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Newly constructed riveted plate girder bridge over the Willamette River, 3 miles south of Newberg, Oreg., on State Route 219. This structure, measuring 1,090 feet and costing \$1,085,000, is located on the Federal-aid secondary system.



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U.S. DEPARTMENT OF COMMERCE

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Correlation Between Chemical and Mortar Bar Tests for Potential Alkali Reactivity of Concrete Aggregates

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Certain siliceous constituents of aggregates can react with cement alkalis to cause abnormal expansion and cracking in concrete. The American Society for Testing Materials (ASTM) mortar bar expansion test is generally considered to be the most reliable method available for determining the potential alkali reactivity of such aggregates. Since this test requires considerable time, a quick chemical test was also adopted by the ASTM in order to obtain a rapid evaluation of concrete aggregates. This investigation was made to determine the degree of correlation between the two test methods.

Fifty-two concrete aggregates were evaluated by means of the rapid chemical test, and the results generally showed good correlation with the data obtained in the mortar bar tests. However, the chemical test does not always give reliable results for some aggregates because of the interference of certain minerals. An empirical division of the chemical test results is proposed which would serve to isolate those aggregates which may contain such interferences, and thereby indicate the need for further tests.

Some aggregates showed a high degree of reactivity in the chemical test but did not produce excessive mortar bar expansion. An additional empirical division of the chemical test results is suggested which would separate chemically reactive aggregates into two groups. Such a separation would eliminate the need of supplementary mortar bar tests on many reactive aggregates. Chemical test results which fall into one such group are indicative of very highly reactive but not necessarily expansion-producing aggregates. Special mortar bar tests are necessary for such materials.

The general chemical test criteria established by this study for separating reactive from nonreactive aggregates agree fairly well with those proposed by the American Society for Testing Materials and the U.S. Corps of Engineers. The present ASTM chemical test method permits two titration procedures for the determination of alkalinity reduction. Neither procedure showed any distinct advantage over the other as a means for evaluating the potential reactivity of aggregates.

BEFORE 1940, most aggregates were usually considered to be chemically inert when used in concrete. However, since that time, it has been clearly demonstrated that certain siliceous constituents, present in some aggregates, can react with the alkalis in cements to cause abnormal expansion and cracking in concrete.

The most reliable laboratory test now available for determining the potential alkali re-

activity of concrete aggregates is considered to be the mortar bar expansion test (1).² This test has been adopted as tentative standard method C227 by the American Society for Testing Materials (2). Because of the time required to obtain reliable information by this test method, extensive investigations have been made to develop a more rapid test. A rapid chemical test was developed by Mielenz and colleagues (3) and was later adopted by the ASTM as tentative standard method C289 (2). Briefly, the chemical test measures the amount of reaction during 24 hours at 80° C. between a 1-normal sodium hydroxide solution and aggregate that has been crushed

² Italic numbers in parentheses refer to the list of references on page 184.

Reported¹ by BERNARD CHAIKEN and
WOODROW J. HALSTEAD, Chemists

and sieved to definite size. At the end of the reaction period, the mixture is filtered and the solution is analyzed to determine its final alkalinity and silica content, the latter arising as a result of the partial dissolution of the aggregate. These values are then used to determine the potential reactivity of the aggregate with alkalis in portland cement concrete.

The reliability of the chemical test was established by the original authors of the method, mainly on the basis of an empirical correlation with mortar bar expansion test results. Unfortunately, many investigators have subsequently reported discrepancies in evaluating concrete aggregates by both test methods.

A search of the literature failed to reveal any additional published work in which a large number of aggregates were evaluated by both test methods. Comparisons of a few aggregates have been reported by several investigators (4, 5, 6). Two other reports (7, 9) showed considerable comparative data, but most of the results were either reported earlier by Mielenz and colleagues (3), or were based upon modifications of the presently used chemical test.

Consequently, the Bureau of Public Roads initiated a study of a large number of aggregates in order to compare the results of both test methods. A specific objective of the study was to determine the degree of correlation between the results of the quick chemical test and the mortar bar test. It was anticipated that the present criteria of the chemical test could be suitably altered or extended in order to make better use of the test results, thereby eliminating the need for further extensive testing of many aggregates.

Conclusions

1. The chemical test for the potential reactivity of aggregates, ASTM C289-57T, generally shows good correlation with the results obtained by mortar bar tests.

2. The dividing line, established by this study for separating reactive from nonreactive aggregates by means of the chemical test,

¹ The authors wish to acknowledge the helpful assistance of Messrs. Donald O. Woolf, Russel H. Brink, and George Markmann. Messrs. Woolf and Brink gave technical advice and supplied physical test data; Mr. Markmann assisted in the preparation of test samples and in the assembling of the mortar bar test data. This article was presented at the 38th Annual Meeting of the Highway Research Board, Washington, D.C., January 1959.

agrees fairly well with those proposed by the ASTM in Specification C 33 and the U.S. Corps of Engineers.

3. Neither the single- nor double-end point titration, permitted by the ASTM chemical test method, offers any distinct advantage in evaluating the potential reactivity of aggregates.

4. Reliable results are not always obtained in the chemical test when certain carbonates or serpentine minerals are present. An empirical division of the chemical test results is proposed which would serve to isolate such interferences, and thereby indicate which aggregates require further study. This procedure would eliminate the need for additional time-consuming tests on many satisfactory aggregates.

5. Highly reactive aggregates may be classified as reactive in the chemical test and yet fail to produce mortar expansion. An additional empirical division of the chemical test results is suggested which would serve to identify such aggregates. It is believed that many such aggregates may actually become dangerous in concrete if diluted with inert aggregates. The proposed division would focus attention upon those aggregates requiring special investigation.

6. A modification of the ASTM main boundary line is proposed as part of the general criteria for chemical tests. The adjusted line conforms more realistically to available data.

Procedure

Chemical tests

Fifty-two concrete aggregate samples were prepared and tested in accordance with ASTM method C289(2), except that the sodium hydroxide solution was prepared by standardizing with standard hydrochloric acid. The acid was standardized against reagent grade sodium carbonate using methyl purple indicator.

The reaction containers used were fabricated from 18-8 stainless steel and fitted with unvulcanized neoprene gaskets. Ethylene glycol was used for the constant temperature liquid bath. Filtration vacuum was maintained at 15 inches of mercury by means of a Cartesian Manostat. Immediately following filtration, the diluted solutions were stored in polyethylene containers and sealed until analyzed.

The reduction in alkalinity was determined by means of both the single- and double-end points given in the ASTM method. The single-end point titration involves a neutralization with standard acid, using phenolphthalein as the indicator, and measures the alkalinity produced by all the hydroxide ions as well as part of the carbonate ions present. In the double-end point procedure, the titration of the same sample is continued to the methyl orange-end point which on calculation gives a measure of the hydroxide ions only.

Dissolved silica was determined colorimetrically with a Beckman DU spectrophotometer using 1 cm. cells. The readings were made at exactly 5 minutes after adding the oxalic acid reagent.

Mortar bar tests

Mortar bar test data had been obtained for these same aggregates over a considerable number of years and, in a large number of cases, well before the development of the ASTM standard method. Consequently, the procedures used in the preparation and storage of the test bars varied somewhat. The details and conditions of test are shown in table 1.

Results

The results of the mortar bar and chemical tests on representative samples of aggregate are tabulated in table 1. Included in the table are the sources and petrographic analysis for most of the aggregates. Table 1 also shows an evaluation of the potential reactivity of the aggregates, based upon the amount of expansion exhibited by mortar bars at an age of 1 year. Expansions of 0.10 percent or more were considered to be indicative of reactive aggregates as suggested by ASTM Specification C33-57T, section 4c(2). Expansions of less than 0.08 percent were considered to be indicative of nonreactive materials, while aggregates having expansions of 0.08 or 0.09 percent were classified as questionable. The latter criteria conform to the established practice of the Bureau of Public Roads.

Some of the aggregates studied consisted of sand and gravel fractions which were evaluated separately in the mortar bar test. However, because of limited amounts of material available after the mortar bar tests, several such companion fractions were combined and prepared as a single sample for the chemical test. In such cases, no special problem arose where the mortar bar data for companion fractions indicated the same degree of reactivity or nonreactivity. In a few cases, however, companion fractions were classified into different categories of mortar bar reactivity. In these few cases, sample Nos. 1, 4, 10, and 11, the result representing the greatest degree of mortar bar reactivity was arbitrarily selected for comparison with the chemical test result.

Table 2 shows only the chemical test results on several additional materials. These were special materials which included carbonate minerals and well-known reactive materials.

The results of the chemical test for each

material, shown in tables 1 and 2, are presented graphically in figures 1 and 2. The units used in both graphs and the method of presentation are in accordance with ASTM method C289(2), and as suggested by the original authors of the chemical test (3). Figure 1 represents the results obtained by using the single-end point in the alkalinity reduction titration, while figure 2 shows the results obtained by means of the double-end point titration.

In figures 1 and 2, each chemical test result is represented by a symbol to denote both the cement used and the degree of reactivity found by the mortar bar test.

Discussion of Test Results

In figures 1 and 2, a solid line was drawn to separate the bulk of the reactive from the nonreactive aggregates. The classification for degree of reactivity was based upon the mortar bar test results as previously described. This manner of separation was proposed by Mielenz and colleagues (3) and is included in a recent revision of ASTM method C-289. Once a line of demarcation is established, it serves as a criterion for future chemical tests in evaluating additional aggregates. Chemical test results to the right of the solid line should be indicative of potentially reactive aggregates, while results to the left of the solid line should represent nonreactive aggregates.

Line of separation

Figures 1 and 2 show two lines of separation. The solid line in each case applies to the data obtained in this study, whereas the dashed line represents other published criteria.

In figure 1, the dashed line is based upon the calculations suggested in ASTM Specification C33-57T(2). Both the dashed and solid lines shown in this graph coincide fairly well; thus the criteria established by this study are in substantial agreement with those suggested by the ASTM specification. A slight spread exists between the two lines in the lower part of the graph. The position of the solid line in this area was dictated by the plotted results of sample Nos. 17, 42, and 34. Sample No. 34 showed a mortar bar expansion of 0.10 percent in 1 year, the minimum value necessary for classifying an aggregate as re-

Table 2.—Results of chemical reactivity test on special materials

Sample identification	Nature of aggregate	Chemical test data ¹		
		Alkalinity reduction (Rc)		Dissolved silica (Sc)
		Single-end point	Double-end point	
A	Limestone (97.2% CaCO ₃).....	mM/L 14.2	mM/L 26.5	mM/L 5.3
B	Dolomite (55.1% CaCO ₃ and 44.3% MgCO ₃)..	363	689	.8
C	Magnesite (Wards).....	405	792	1.5
D	Magnesite (Pennsylvania).....	440	855	.5
E	Opal (Virgin Valley, Nev.).....	325	369	778
F	do. ²	246	296	949
G	Siliceous magnesium limestone (Calif.).....	218	278	557
H	Pyrex glass cullet.....	120	175	747

¹ ASTM Method C 289-57T.

² Mix of 50 percent opal and 50 percent Ottawa sand.

active. In addition, mortar bars of this material were stored at 70° F. and failed to show significant expansion at 1 year (11). Since sample No. 34 is a borderline aggregate, the location of the solid line in the extreme lower portion of figure 1 may not be precise, as it is based upon the results of this aggregate alone.

The dashed line in figure 2 represents the criteria used by the U.S. Corps of Engineers (10), and here again there is good agreement between the position of both lines, except for the same type of spread in the lower part of the graph as noted in figure 1.

Single- versus double-end point titrations

An inspection was made of both graphs in order to determine whether the single- or double-end point titration procedures offered any distinct advantage. The U.S. Corps of Engineers apparently prefers the double-end point titration (10), while the Bureau of Reclamation and many other laboratories rely mainly on the single-end point titration. Table 3 lists the aggregates that failed to show good correlation between the chemical and mortar bar tests. This table shows that there is no difference between figures 1 and 2 in either the identification or number of such discrepancies. Therefore, no substantial advantage is evident for either type of titration procedure.

Comparison of Chemical and Mortar Bar Test Results

In some instances, the empirical classification of the aggregates into reactive and non-reactive materials, based upon the chemical test results, differs from the classification obtained by means of the mortar bar test. Specific consideration of samples showing such differences is of interest.

In figure 1, a total of 29 symbols appear to the left of the solid line. Of these, 24 are clear symbols denoting nonreactive aggregates and consequently are properly located in the graph. Three other aggregates in this area (Nos. 1, 35, and 42) were found to be of questionable reactivity, with mortar expansions of 0.08 percent at 1 year. However, the chemical test results of all three samples are located close to the dividing line where questionable aggregates might be expected.

Only two aggregates (Nos. 12 and 31), classified as either reactive or questionable, are located well to the left of the dividing line. There is no question as to the potential reactivity of sample No. 12, since this aggregate was found to cause significant mortar expansion even when combined with a medium alkali cement (11). However, both of these aggregates contain limestone or dolomitic constituents which, as discussed later, may invalidate the chemical test results.

A total of 23 symbols appear to the right of the solid line in figure 1. It would be expected that all aggregates located in this area should be reactive. However, seven samples were shown to be nonreactive by the mortar bar test. The identification of these aggregates is given in table 3. Sample Nos.

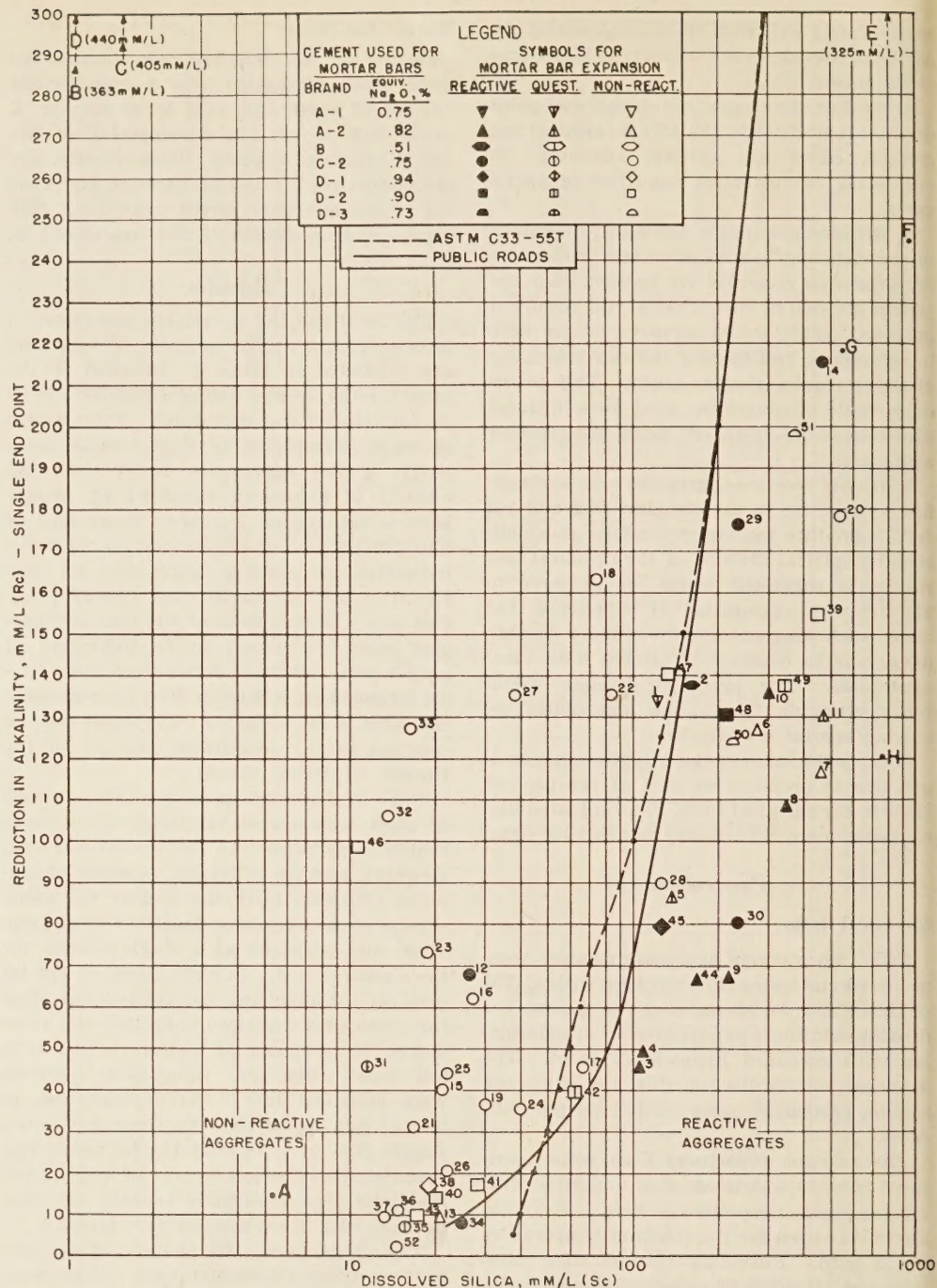


Figure 1.—Results of chemical test for reactivity of aggregate obtained by means of single-end point titration.

39, 50, and 51, included for information purposes only, may be eliminated from this discussion since they were not tested in mortar bars, but rather as concrete specimens prepared from gravel size aggregate.

Therefore, only 4 samples (Nos. 6, 7, 20, and 28) are considered as limiting the degree of correlation between the chemical and mortar tests in this area. However, it was noted that all four samples exhibited some degree of mortar bar reactivity in supplementary tests. Sample Nos. 6, 7, and 20 caused considerable mortar expansion when the mortar contained an admixture of sodium hydroxide, while sample No. 28 began to show a questionable degree of reactivity in the mortar bar test at an age of 2 years (11). These latter findings lend support to the chemical test data

obtained for these aggregates. It serves to emphasize that while some aggregates may be potentially reactive and are so characterized by the chemical test, these same aggregates may not be capable of producing dangerous expansion in the standard mortar bar test. Discrepancies of this sort are discussed more extensively in the following section.

Factors Affecting Correlation of Test Methods

Carbonate interference

The two aggregates (Nos. 12 and 31), which appeared as distinct discrepancies in the area to the left of the solid line in figure 1, were classified petrographically as containing certain carbonate constituents. Differential

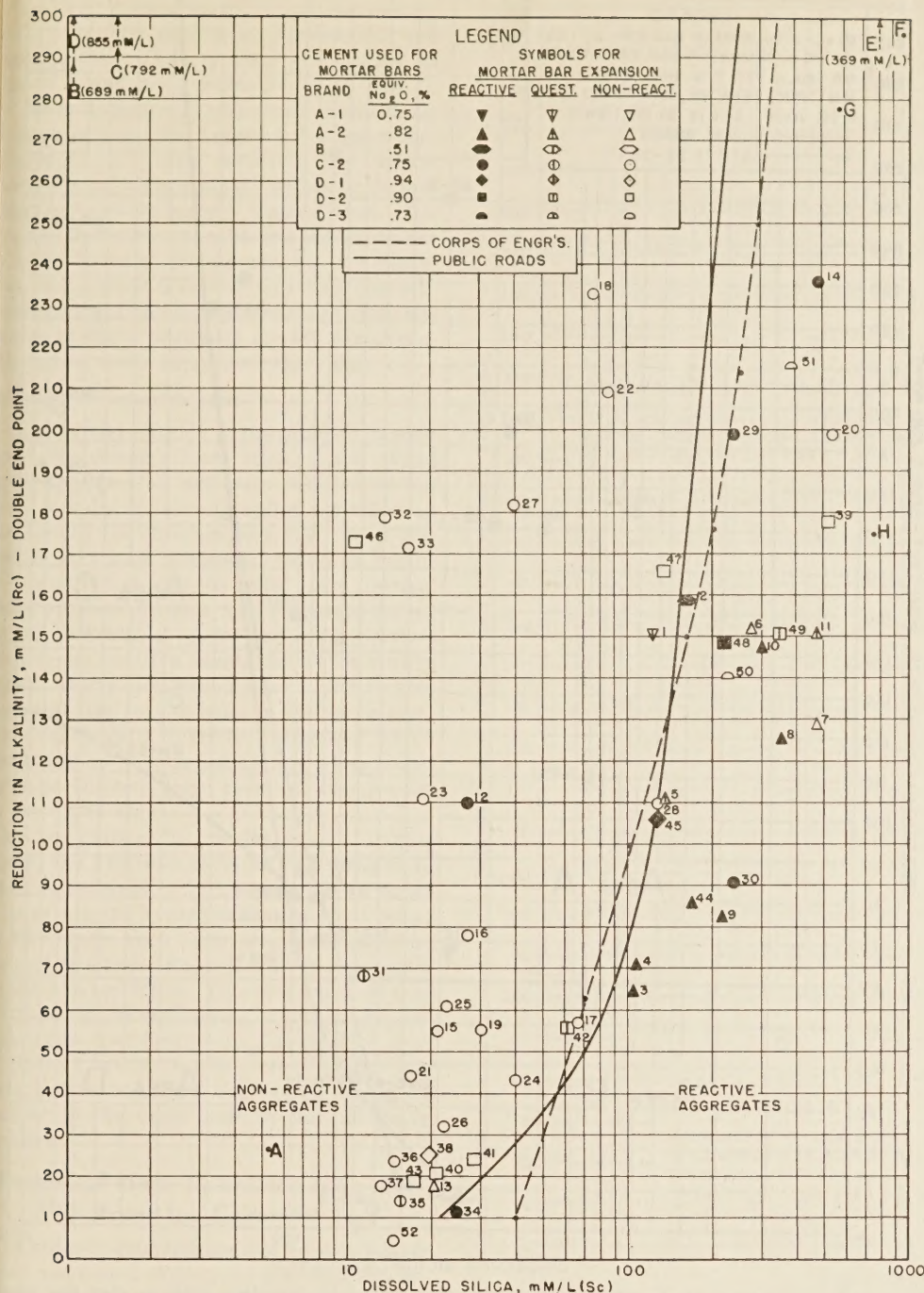


Figure 2.—Results of chemical test for reactivity of aggregate obtained by means of double-end point titration.

Table 3.—Identification of aggregates which showed poor correlation between chemical and mortar bar tests (selected by inspection of figures 1 and 2)

Position of plotted chemical test results	From figures 1 and 2	
	Aggregate identification	Number of samples
Left of solid line:		
Reactive aggregates ¹	No. 12 ²	1
Questionable aggregates ¹	Nos. 1, 31, 35, 42	4
All aggregates		29
Right of solid line:		
Nonreactive aggregates ¹	Nos. 6, 7, 20, 28, 39, 50, 51 ³	7
All aggregates		23

¹Based on mortar bar test results and criteria shown in text and in table 1.

²Sample contains 16 percent dolomite as determined by DTA methods.

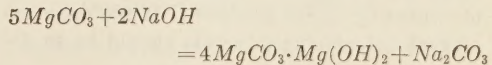
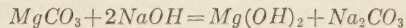
³Three samples (in italics) were not tested in mortar bars, but as concrete specimens prepared from gravel size aggregate.

thermal analysis showed that sample No. 12 contained both calcite and dolomite, while sample No. 31 contained essentially calcite without dolomite and magnesite. Mielenz and Benton (9), as well as Swenson (6), have indicated that dolomites and perhaps even limestones may at times cause significant interference in the chemical test. It was demonstrated that although certain opal-dolomite mixtures were found to be deleterious by mortar bar tests, the chemical test results gave false information concerning the reactivity of the mixture (9). Such misleading information was attributed to the interference of dolomite and other minerals such as magnesite, siderite, and serpentine. When present along with reactive constituents, these minerals produce greater reductions in alkalinity (Rc) and lower dissolved silica values (Sc) in the chemical test than would otherwise be obtained. The net effect of such interference is to shift the chemical test data both to the left and upward in figure 1, and thereby, at times, yield misleading information concerning a truly reactive aggregate.

A separate study of several carbonate materials was made in order to understand more fully their possible effects on the chemical test. Table 2 shows the chemical test results obtained on four carbonate minerals. Relatively pure limestone, dolomite, and two magnesites were tested, and the tabulated results were plotted in figures 1 and 2 as points A, B, C, and D.

As seen in table 2 and figure 1, the limestone (A) did not cause any significant alkalinity reduction, while the dolomite (B) and both magnesites (C and D) produced unusually large alkalinity reduction values (Rc). It would appear from these results, that pure limestone (calcium carbonate) does not react significantly with the sodium hydroxide solution used in the test, while dolomite and magnesite do. Furthermore, the ratio of the double- and single-end point values of Rc for both dolomite and magnesite, and the absolute difference between the values, suggest the formation of considerable quantities of weakly basic material in solution during the test.

The sodium hydroxide solution was analyzed in each case after the test was completed. No significant amounts of calcium or magnesium were detected nor was there any apparent decrease in the concentration of sodium below the level of the initial test solution. In the case of dolomite and magnesite, it was concluded that the test solution had lost some hydroxide ions in exchange for an equivalent amount of carbonate ions. The latter material accounts for the presence of a weak base after test. From these considerations, two possible equations can be written for the reaction between magnesite and the sodium hydroxide solution. Such a reaction may be represented by either of the following equations:



Such reactions appear reasonable from a consideration of the solubility products of magnesium carbonate (2.6×10^{-5} at 12°C.) and magnesium hydroxide (1.2×10^{-11} at 18°C.) (12). The latter material, a possible end product in the reaction, is much more insoluble in water than is magnesium carbonate, the reactant. The reaction between dolomite and the test solution undoubtedly is similar to that involving magnesite.

No special study was made to explain the depressed values of dissolved silica (Sc) caused by the presence of dolomite or similar materials (9). The following thoughts are offered as possible explanations of this effect: (a) The large reduction in alkalinity caused by such minerals may so reduce the alkalinity of the solution as to minimize its attack upon reactive forms of silica; (b) the formation of insoluble magnesium hydroxide or the basic carbonate may act as a protective film in limiting the attack of the solution upon reactive forms of silica; or (c) dolomite or its decomposition products may react with dissolved silica to form insoluble silicates and thereby remove silica from solution.

Elimination of carbonate interference

Efforts by Mielenz and Benton to eliminate carbonate interference in the chemical test by prior treatment of the aggregate with hydrochloric acid have not given satisfactory results (9). These authors have suggested that an analysis of the change in concentration of sodium and potassium rather than the reduction in alkalinity (Rc) would probably remove a large part of the spurious effects induced by such carbonates. While such a procedure would eliminate the falsely high Rc values, it would not eliminate the low Sc values produced by this interference.

Continued study of the "carbonate effect" on the chemical test is necessary in order to eliminate its interference. Because of such interference, a recent revision of the present ASTM chemical test recommends petrographic examination as a supplement to the chemical test data. This recommendation would involve a time-consuming petrographic examination of many aggregates, which on the basis of chemical tests would fall to the left of the dividing line in figure 1.

However, a more expedient alternative is suggested here. It has been shown that the "dolomite interference" is characterized by rather high Rc values in the chemical test. A level of Rc might be established so that all results below this level and to the left of the main dividing line may safely be considered nonreactive aggregates and, consequently, no further testing would be necessary. Results above this Rc level would require additional study by chemical, petrographic, X-ray, or differential thermal analysis (DTA) methods in order to detect the presence of either interfering or reactive minerals. The presence of small amounts of dolomite, magnesite, or calcite may be detected by spot tests or differential thermal analysis, while serpentine is conveniently identified by X-ray analysis. If the presence of such interfering minerals should be so de-

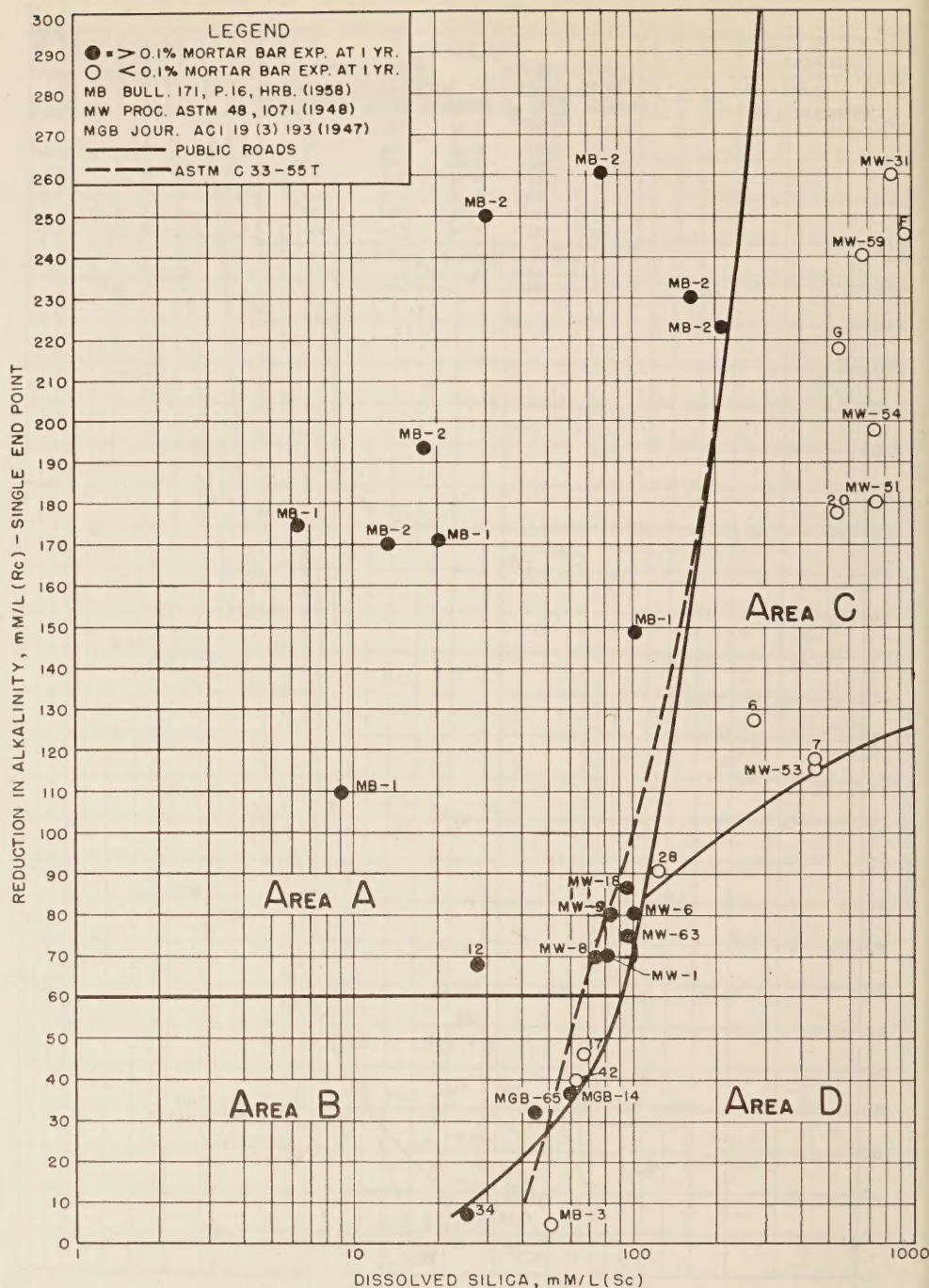


Figure 3.—Location of chemical test results that failed to correlate with mortar bar tests or other criteria.

tected, mortar bar tests would then be necessary in order to evaluate more fully the potential reactivity of the aggregate. The proper location of a dividing line is discussed later.

Highly reactive materials

In addition to the results of this study, other investigators have found that many chemical test results falling to the right of the main dividing line fail to correlate with mortar bar test results (4, 7, 8). Both Sprague (8) and others (7) have attributed such discrepancies to the fact that certain aggregates contain such a large proportion of reactive constituents in relation to available cement alkalis that they fail to produce mortar bar expansion. This is a consequence

of the "pessimism proportion"³ of the aggregate being materially exceeded.

It is well known that certain opals and siliceous magnesium limestones are highly reactive and are so characterized by the chemical test. Yet when these materials are present in large proportions, they actually cause little or no expansion in the mortar bar test (11, 13, 14). It was shown by Woolf that 1 or 2 percent of opal combined with an inert material will produce dangerous mortar bar expansion, whereas much larger amounts of opal will cause insignificant expansion for a given cement alkali content (16).

Table 2 shows the chemical test results on

³ Defined by T. E. Stanton (13) as that proportion of reactive constituent in an aggregate at which maximum mortar bar expansion occurs.

several well-known reactive materials; namely opal, siliceous magnesium limestone, and pyrex glass. These results are plotted in figures 1 and 2 as points E, F, G, and H. In figure 1, points F and G are particularly interesting since they approach the area where discrepancies (clear symbols) are shown. Point F represents a blend containing 50 percent opal and 50 percent Ottawa sand. McConnell and others have shown that a similar mixture failed to produce mortar bar expansion (15). Point G represents 100-percent siliceous magnesium limestone which Stanton has shown to cause negligible mortar bar expansion when tested in its entirety (13).

Division of reactive area

As a partial solution to the problem of such recurring discrepancies, Mielenz and Witte have suggested an arbitrary division of the area containing the aggregates classified as reactive by the chemical test (7). They suggested that such a division would be useful in interpreting the chemical test results. Their proposed division would separate the reactive area into an upper and lower portion. Chemically reactive aggregates which always caused mortar expansion were found to give chemical test results lying in the lower area, while chemically reactive aggregates which did not cause mortar expansion, when tested in their entirety, gave chemical test results in the upper area.

On the basis of the results obtained in this study, the proposed dividing line appears to have considerable merit. However, in order to separate the apparent discrepancies found in this study, such a dividing line should be located at a lower Rc level than that proposed by Mielenz and Witte. This study showed that all four of the aggregates having major discrepancies and located in the reactive area of figure 1 had Rc values of 90 or more. The data points for these aggregates should be located in the upper area of any arbitrary division.

Proposed Main Boundary Line and New Areas for Chemical Test

In order to establish acceptable area boundaries mentioned earlier, all discrepancies between chemical test and mortar test data were evaluated. Figure 3 shows the location of all such data points both from this study and the available literature. In addition, data related to the proper location of a main boundary line were similarly plotted. Results taken from the literature are partly identified in figure 3 by a letter symbol to denote the source.

Location of main boundary line

The data chosen for presentation with respect to the main boundary line are shown in the lower third of figure 3, and generally fall within the area between the two dividing lines.

The different positions of the two lines may be partially explained on the basis of the cements used in the mortar bar tests. The high-alkali cements in this study had an equivalent alkali content of less than 1.0 percent. The data given by Mielenz and colleagues

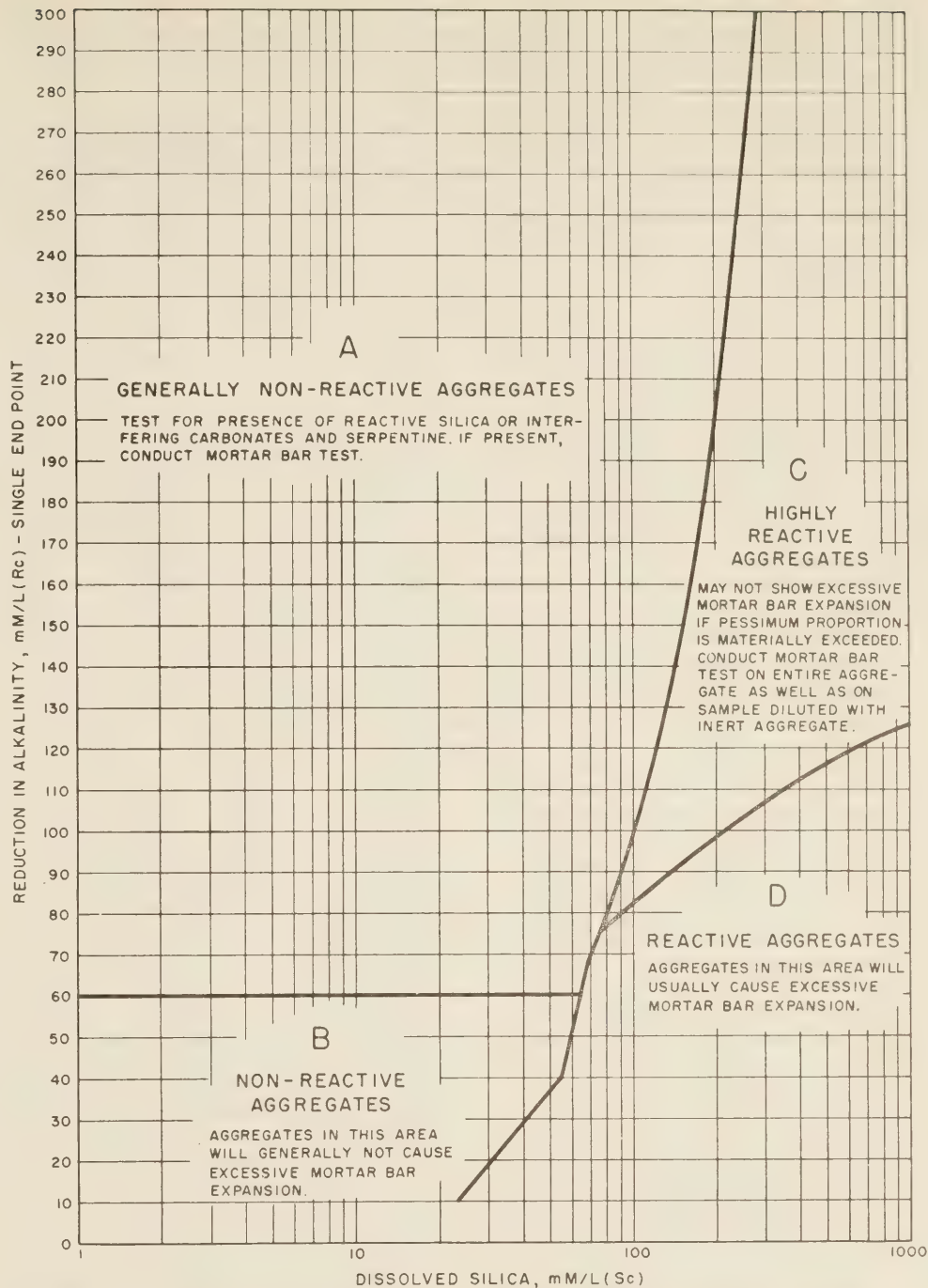


Figure 4.—Evaluation of results of chemical test for potential reactivity of aggregates.

(3, 7), which provided the major basis for the ASTM line, were obtained with a cement of 1.38 percent equivalent alkali. Such differences in cement alkalies could cause a borderline aggregate to produce expansions less than 0.1 percent in one case and greater than 0.1 percent in the other. This, of course, would affect the position of any boundary line which is empirically drawn to separate reactive from nonreactive aggregates.

In order to establish a suitable boundary line for general use, the following considerations were taken into account: (1) The present ASTM line occupies a more conservative position with respect to the Bureau of Public Roads line in that there would be less danger of the nonreactive area containing chemical results of unsafe aggregates. (2) Samples MW-1, 8, 9, 18, 63, and MGB-14 were reported

as causing deterioration in field concrete (3). Consequently, such aggregates should properly appear to the right of any dividing line. (3) Sample 42, in this study, showed 0.08 percent expansion at 1 year and could conceivably have given 0.1 percent expansion with a higher alkali cement.

In view of these considerations, the present ASTM line is preferable, except the lower portion should be adjusted to indicate clearly the deleterious nature of aggregates 34 and MGB-65. This can be accomplished by changing the slope of the lower portion of the ASTM line. For Rc values from 10 to 40, the formula for the adjusted line is as follows:

$$\log Sc = 1.2553 + 0.01213 Rc$$

Figure 4 shows the proposed change in the

ASTM line between $R_c=40$ and $R_c=10$. All available published data were reviewed with respect to this change, and only one conflict was observed (aggregate MB-3). This aggregate, shown in figure 3, was a synthetic opal-quartz mixture with 0.1 percent opal which failed to show excessive mortar expansion (9). Its position is anomalous, even with respect to the original unchanged ASTM line, and no explanation of its position is attempted here.

Areas A and B

The left portion of figure 3 was evaluated, disregarding the aggregates just mentioned which had to do with the location of the main boundary line. It can be seen that all discrepancies (aggregates which produced mortar expansion) are in the upper portion of this area. No discrepancies appear below an R_c level of 60; therefore this value can be used to divide the left portion of figure 3 into two areas, A and B. In each case, the anomalous result shown in area A represents an aggregate which contains constituents which are known to interfere in the chemical test. Sample 12 contained dolomite; MB-1 contained various dilutions of opal in dolomite; and MB-2, various dilutions of opal in serpentine.

On the basis of the available data, area B should be clearly indicative of safe aggregates. Chemical results appearing in this area should be reliable and no further tests are required. In general, aggregates that appear in area A will also be safe; however, the presence of interfering minerals such as dolomite and serpentine may at times result in an unsafe aggregate appearing in area A. Consequently, all aggregates which fall into area A should be further evaluated for the presence of either reactive constituents or interfering minerals. In doubtful cases, mortar bar tests should be made.

Areas C and D

The right portion of figure 3 was similarly evaluated, again disregarding the results which had to do with the main boundary line location. All discrepancies appear in the upper portion of this area. Consequently, the right portion of the graph may be divided into two areas, C and D. No discrepancies appear in area D; therefore, chemical results which would appear in this area should be indicative of unsafe aggregates which are capable of producing excessive mortar expansion. No further tests on such aggregates would be necessary.

Area C contains numerous aggregates which did not produce mortar expansion (clear symbols). This area also contains unsafe aggregates as is evident in figure 1. Consequently, the purpose of area C is to focus attention upon aggregates requiring special investigation as a result of the chemical test data.

It is interesting to note that all aggregates in the upper portion of area C, figure 3, produce excessive mortar expansion when diluted to very small concentrations with inert material. Sample F is a 50-50 opal-Ottawa sand

mixture; G and MW-59 are undiluted siliceous magnesium limestones from California; and MW-31 is an undiluted rhyolite tuff from Colorado.

On the other hand, several of the aggregates in the lower portion of area C, namely MW-51, 53, and 54, will cause excessive and maximum mortar bar expansion when present in about 20-percent concentrations with inert material (7). It may be that aggregates 6, 7, and 20 of the present study are similar and might have shown unsafe expansion upon dilution.

Consequently, chemical results which appear in area C suggest a highly reactive material, but do not necessarily preclude that all such materials will produce excessive mortar expansion. The ability to produce unsafe expansion will depend largely on the proportion of reactive constituent present. Therefore, all aggregates which yield chemical data in area C should be carefully examined by mortar bar tests. They should be tested in dilutions of 5, 10, 20, and greater percentages with inert materials, as well as in their entirety. This procedure is especially important where local shortages or special conditions require that aggregates from different sources be blended to make concrete.

Summary of New Areas and Boundary Line

Figure 4 indicates the significance of each of the proposed areas in evaluating chemical test results. It also shows the main ASTM boundary line, which has been modified at the lower end as discussed earlier.

The location of all boundary lines is, of course, empirical in nature and dependent exclusively upon available mortar bar and chemical test data. The location of such boundary lines may require minor adjustments as new data become available in the future. In addition, the limited reproducibility of the mortar bar and chemical tests, as reported by the U.S. Corps of Engineers (10, 17), always raises some doubt as to the precise location of any dividing line. Consequently, marginal chemical test results, especially near the main boundary line, should be interpreted with some reservation.

Nevertheless, the present criteria shown in figure 4 should broaden the usefulness of the quick chemical test and, in many cases, eliminate the need for supplementing the chemical test with a long-term mortar bar test program or tedious petrographic analysis.

In one respect, the chemical test offers a unique advantage over the exclusive use of the mortar bar test. This applies to aggregates which appear in area C of figure 4. The engineer may obtain a false sense of security from a satisfactory (safe) mortar bar test result on such aggregates. However, it is possible that some of these materials will cause deterioration in concrete if blended with other relatively inert aggregates. The chemical test result would indicate the need for more careful evaluation of such materials.

(For conclusions, see pages 177 and 179.)

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(Continued on page 190)

Home-Interview Traffic Surveys and Related Research Activities

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported¹ by JOHN T. LYNCH, Chief,
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Interarea Travel Formula

The interarea travel formula for primary trips which is being evaluated at this time is, in general form, as follows:

$$T_{ij} = \left(P_i \frac{A_j}{\Sigma A} + P_j \frac{A_i}{\Sigma A} \right) \frac{K}{D_{ij}^n}$$

Where:

T_{ij} = The number of primary trips between zones i and j , that is, with one end at home.

P_i and P_j = The number of primary trips produced in zones i and j by residents of those zones.

A_i and A_j = The number of primary trips attracted to zones i and j by nonresidents plus intrazone trips by residents of those zones.

ΣA = The number of primary trips attracted to all zones.

D_{ij} = The distance between zones i and j , generally expressed in terms of travel time.

K = A constant and n an exponent, both to be determined empirically from the analysis.

After K and n have been calculated, the application of the formula will probably result in a different number of trips attracted to, and trips produced in each of the zones than was originally estimated from the land-use data. A series of successive approximations or some other method will then have to be used to make the number of trips in and out of each zone, calculated from the formula, agree with the original estimates of trip-ends.

This formula and a similar one for secondary, or non-home-based trips, are being evaluated by the Bureau of Public Roads with the aid of an electronic computer, using data from the Washington metropolitan area 1948 and 1955 O and D surveys. For trips produced and trips attracted (the P and A factors), actual survey values are used; the test is to determine whether a formula of this general form will produce reliable results, and if so, what values of K and n are applicable to different situations.

The test is being made by trip purpose, as it is expected that the value of the time exponent n will vary for different purposes. It is thought, for example, that travel time will be more of a deterrent for shopping trips than

[T HAS now been 15 years since the home-interview method of making origin and destination surveys in urban areas was developed. The first surveys by this method were made during World War II, when conditions were quite abnormal. The method has now been used in 126 urban areas and repeat surveys have been made or started in 10 of these. During 1958, surveys were begun in the following areas: Vallejo, Calif., Champaign-Urbana, Ill., Topeka, Kans., Minneapolis-St. Paul, Minn., Charlotte, N.C., Pittsburgh and Reading, Pa., and El Paso, Tex.

The Minneapolis-St. Paul, Charlotte, and Reading surveys were repeat studies. In addition, repeat studies were being organized for Denver, Philadelphia-Camden, and Nashville, and were under consideration for Atlanta, New Orleans, Tucson, and Honolulu.

The field methods and the information obtained have been changed very little since the earliest surveys. A few items have been added or altered, however, to aid in the study of trip projection. Present emphasis is on the improvement of analysis methods. It was recognized in the beginning that the most difficult problem was to find a satisfactory method of forecasting travel. It was thought that fundamental relations could be established which would permit the determination of the future travel pattern from projections of land use, population, and economic factors. For this reason information was obtained in the interview on such items as size of family, number of cars owned, occupation and industry, mode of travel, and purpose of trip.

Development of Land-Use Data

One important addition that has been made in the present practice is the determination of land use or the character of establishment, at the beginning and the end of each trip. This is done in considerable detail; for example, a shopping trip made to a food store is differentiated from one made to a clothing store, and a social trip made to a residence is recorded differently from a recreational trip made to a theater. Such detail requires about 60 land-use classifications. This detailed

classification of land use at trip-ends was made first in the Chicago study, which is nearing completion, and is being made in most of the surveys that have been started during the past year.

The purpose of the detailed land-use information is twofold: First, the number of trips attracted to each class of development or establishment can be determined and related to some measurable factor such as population, automobile ownership, employment, dollar sales, or area; secondly, the effect of distance or travel time on trip attraction for each class can be studied. These are the important factors which enter into any interarea travel formula such as the so-called "gravity model."

Of course, it will not be practicable to forecast land use in any such detail. However, analysis of the data will aid in evaluating the variability of the trip attraction and distance factors for different land uses, in testing the form and possible accuracy of an interarea travel formula, in improving the zoning by grouping land uses of similar trip attraction characteristics to the extent practicable, and in estimating the factors for zones on the basis of approximate anticipated distribution of land uses within the zone.

Throughout the past 10 years, much research has been conducted on trip production and trip attraction in relation to land use. The results of many such studies have been published by the Bureau of Public Roads and the Highway Research Board (1-8).² Difficulties have been encountered because of mixed land uses within zones and lack of precise knowledge as to the character of the establishment at the ends of each trip. Progress has been slow because of the mass of detailed data to be analyzed and the shortage of personnel. However, some fairly reliable factors have been developed for trip production by purpose for residential zones, and for the effect of distance on shopping trips. What remains to be done is to test and evaluate various interarea travel formulas that have been proposed and to develop, if possible, statistically reliable factors for use in these formulas. Because of the availability of electronic computers, considerable progress is being made in this direction.

¹ This paper was presented by Mr. Lynch before the Committee on Origin and Destination Surveys at the 38th Annual Meeting of the Highway Research Board, Washington, D.C., January 1959.

² Italic numbers in parentheses refer to the list of references on p. 186.

for work trips. Also, it is expected that n will be smaller for trips to the central business district than for other trips.

In this test, the travel times reported in the interviews are being used. This part of the program has been completed and some preliminary runs have been made. The travel times appear to be reasonably consistent; the relation between peak hour and offpeak hour times, for example, are about as would be expected. It is necessary, however, to smooth out the reported times because of the small number of reported trips between some pairs of zones, and also because of the tendency to report times in even 5- or 10-minute intervals. This is being done through a study of time-distance relationships for pairs of zones similarly situated.

Data from the 1955 survey are being analyzed first. Subsequently, it is planned to use the data from the 1948 survey to see whether the values of K and n change over a period of time.

A similar test of this formula is being made by the Ohio Department of Highways, using data from the Dayton and Toledo surveys. These were controlled postcard surveys, and neither trip purpose nor travel time was obtained. It will be necessary, therefore, to confine the analysis to the total number of trips for all purposes, and to determine travel time from time runs. The survey will have special value, however, in permitting a comparison of the results for the two cities, as well as with those for Washington, D.C.

A program is being developed by the Bureau of Public Roads for testing the application of an interarea travel formula to travel between cities or counties. The States have supplied a large amount of data for this program from external surveys around urban areas throughout the country. Factors currently being considered are population, median income, automobile registrations, and persons employed in eating and drinking establishments and in hotels and other places of lodging. The last named factor is thought to be of special importance in connection with recreational travel. Trip production by residents and trip attraction for nonresidents will be considered separately as in the case of the formula that has been described. An adjustment of airline distance to highway distance will probably be used instead of travel time.

Electronic Computer Programming

Last year, Brokke and Mertz reported on a test of the Fratar, average-factor, and Detroit growth formulas for projecting the future urban traffic pattern from the present one (9). They found that the three formulas gave results of equal accuracy, but that the Fratar method required fewer iterations than the others. The standard errors for zone-to-zone movements were naturally high because of the very small number of trips between many pairs of zones. Zones were grouped, therefore, to give substantial trip volumes between each of the groups. When this was done, the standard errors were not unreasonably high. Since the grouped zones are much

larger than would be used in practice, a different kind of test is thought to be necessary. What engineers actually wish to know is how accurate the results will be when a large number of zone-to-zone movements are accumulated on a freeway. It is planned, therefore, to accumulate the trips across grid lines and to determine the standard errors for sections of the grid crossed by substantial volumes of trips. This project is still in the planning stage, but it is hoped that it can be carried out during 1959.

A program to apply the Fratar formula has been written for a 705 IBM computer by the Bureau of Public Roads with the assistance of the National Bureau of Standards. This program has been used to project the future zonal traffic interchange from the 1955 data for the Washington area.

In the Chicago area transportation study, a program has been developed for the IBM 704 computer for assigning zone-to-zone traffic movements to a complete urban highway network, based on a minimum travel time concept (10). A similar program, based on somewhat different concepts, is being developed for the Detroit area traffic study.

A program to accomplish the same purpose has been written for the IBM 704 computer by the General Electric Computer Division for the Regional Highway Planning Committee of the Washington metropolitan area, with the assistance of Bureau of Public Roads personnel. This program, based on a time ratio rather than on a minimum travel time concept, will be described in a subsequent issue of PUBLIC ROADS magazine.

Programs similar to those mentioned undoubtedly can be developed for other computers of equivalent capacity and characteristics.

Measurement of Mass-Transit Usage

One of the most important accomplishments of 1958 was the developing and testing of what seems to be a satisfactory formula for estimating proportionate mass-transit usage. There has been a lack of knowledge in this field that would permit the distribution of future trips between mass transit and private automobiles with any degree of assurance. The formula involves a number of factors and is based on data obtained in part from the O and D surveys, in part from the transit companies, and in part from planning commissions. Full cooperation of such agencies in more than 20 cities has made the accomplishment possible.

The formula, as so far developed and tested, seems to predict the proportion of the trips made by mass transit in different cities with a surprising degree of accuracy. Preliminary tests indicate that it is also applicable to the prediction of mass-transit usage in different sections of the same city, but this will be further tested in 1959 as additional information becomes available.

In some of the O and D studies now being started, provision is made to obtain information on availability and cost of parking, and on walking time between place of parking or transit and origin or destination. It is

thought that factors may be developed from such information that will improve the transit-usage formula.

Future Prospects

On the whole, the prospects for important research accomplishments during 1959 in the field of O and D studies seem bright. The number of people engaged in such research in the Bureau of Public Roads remains small but the availability of electronic computers is speeding up the work. Furthermore, with greatly increased highway planning surveys, funds, the States are showing increased willingness and desire to engage in research that will aid them in forecasting traffic volume and patterns. In the St. Paul-Minneapolis study, for example, \$75,000 has been programmed for research. In addition to the interarea travel formula test that has been mentioned, the Ohio Department of Highways is planning to undertake some research on the diversion, generation, and growth of traffic following the construction of rural freeway sections. Highway impact studies are under way in many States; in some cases the work is being done by State personnel and in others, by university staffs. The North Carolina State Highway Commission is conducting several traffic research studies, through their highway planning division and through university cooperation.

In most of the current urban O and D studies, continuing organizations are being set up or planned. These organizations will, in many cases, carry out research. In Detroit, for example, continuing research is being planned through an agreement with Wayne University.

Considering all of these factors, it is reasonable to hope that the analysis and projection of traffic patterns will be placed on a sound scientific basis within a few years.

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Travel to commercial centers of the Wash-

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Pretreatment of Soils and Clays for Measuring External Surface Area by Glycerol Retention

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported by EARL B. KINTER and SIDNEY
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IN TWO previous reports by the authors (1),¹ a method was presented for determining the surface area of clays and soils from the weight of sorbed glycerol retained by specimens after extended heating at 110° C. in the presence of free glycerol vapor. For the particles of clay minerals such as kaolinite, only external surfaces are available for sorption; but for expanding minerals such as montmorillonite, the glycerol is also retained on internal surfaces between the unit layers of the crystallites.

This internally sorbed glycerol forms what has been called a "monointerlayer" complex, that is, a monomolecular layer or film between two planar clay surfaces. Each molecule of glycerol in such a complex covers twice as much clay surface as does a molecule sorbed on external surfaces. The factor for conversion of a given weight of the internal film to values of surface area is therefore twice as large as for an equal weight of a similar film on external surfaces. For the general use of the glycerol retention method with soils and clays, it is thus necessary to evaluate separately the glycerol retained on the two types of surface, unless it is known that even trace quantities of the expanding minerals are not present.

Procedure

The procedure employed for this purpose was adapted from that used by Dyal and Hendricks (2) in connection with the sorption of ethylene glycol by clays. As applied to the sorption of glycerol, this procedure consisted of determining separately the glycerol retained by an untreated sample and by a sample of the same material previously heated to 600° C.

It was assumed that the heat treatment would irreversibly collapse expanding mineral lattices so as to seal the internal surfaces against subsequent penetration by glycerol and permit sorption on only the external surfaces; and since the untreated material would retain glycerol on both types of surface, the difference in retention between the two samples was accordingly attributed to internal surfaces.

For this procedure to result in a realistic measure of the surface area of a given sample, it is obvious that the heat treatment must completely and irreversibly collapse any expanding layers present, but must not, at the same time, appreciably alter the original external surfaces of constituent particles. However, depending on the composition of the sample to be studied, one or both of these conditions may not be fulfilled. For example, in the first report cited (1), the authors showed that with certain montmorillonites the 600° C. heat treatment is not sufficient for irreversible collapse. These montmorillonites would, therefore, still sorb glycerol internally to some extent, and thus the amount attributed to external surfaces would be too high. On the other hand, temperatures above 600° C. cannot be used to ensure irreversible collapse because evidence exists that this would sometimes severely reduce external surface areas (3).

The heat treatment at 600° C. may also affect minerals other than montmorillonite. Nitrogen adsorption measurements (4) have shown that the surface area of kaolinite was reduced by about 20 percent on heating to this temperature. The present authors had noted that a substantial reduction in glycerol retention is also produced with illite, but with this mineral the reduction was ascribed to the irreversible collapse of expandable layers contained within the illite particles. The external surface of vermiculite, on the other hand, has been shown to increase threefold on heating to 600° C. (5).

It is also to be expected that heating would result in pronounced increases in surface area with certain of the common accessory "non-clay" minerals of soils and clays. For example, Russell and Cochran (6) observed that the surface area of an aluminum trihydrate closely similar to natural gibbsite increased from about 10 to 425 square meters per gram ($\text{m}^2/\text{g}.$) on heating to 400° C.; and a subsequent decrease to about 280 $\text{m}^2/\text{g}.$ occurred on heating to 600° C. Similar increases should be anticipated for the crystallized hydrous iron oxides, goethite, and lepidocrocite, although conversely, heating an amorphous ferric oxide gel to 600° C. has been shown to reduce the specific surface area from about 250 to 50 $\text{m}^2/\text{g}.$ (7).

Many soil properties important in highway work, such as plasticity and volume change, are caused by the interaction of soil particle surfaces and water. Similarly, interactions between particle surfaces and chemical additives are involved in soil stabilization. In a given soil, both types of interaction are largely controlled by the amount of surface area possessed by the soil particles.

A convenient and accurate method for measuring the surface area of soil materials should therefore be quite useful in studying and evaluating soil properties for highway purposes. Such a method was developed by the Bureau of Public Roads in 1956. The present article reports results of more recent work concerning a refinement of the original test procedure.

In general, then, the value of 600° C. heating is doubtful at best for materials of such varied composition as soils and clays. In the search for a suitable substitute, it was thought that a chemical pretreatment might be found by which the entrance of glycerol molecules into the interlayer spaces of expanding minerals could be prevented, without either interfering with normal adsorption on external surfaces or reducing or increasing the external surface area of any of the minerals present.

Since exchangeable cations are known to be distributed between the unit layers of montmorillonite, attention was first given to cation exchange treatments. None was found suitable, however, until, as suggested by the work of Greene-Kelly (8), the triethylammonium cation (TEA) was tried. Greene-Kelly had reported that a montmorillonite saturated with TEA exhibited a consistent basal spacing of approximately 13.3 angstroms (1 A.=one hundred-millionth of a centimeter), regardless of whether the sample was air-dried, immersed in water, or immersed in glycerol. In trials with several other montmorillonites, the present authors confirmed these observations and also noted that the 13.3 A. spacing was not altered when specimens were dried at 110° C. or washed with water or acetone.

This apparently very stable spacing indicates a distance of about 3.8 A. between the montmorillonite layers, which would naturally exclude glycerol molecules since they

¹ Italic numbers in parentheses refer to the list of references on page 190.

are about 4.5A. in diameter. Preliminary trials with montmorillonites indicated that saturation with TEA definitely blocked the penetration of glycerol into the interlayer spaces, and apparently did not interfere with normal adsorption on external surfaces. The investigation of the use of TEA saturation as a replacement for the 600° C. heat treatment forms the basis of the present report.

Conclusions

The evidence presented indicates that when soils and clays are heated at 600° C. as a pretreatment for the collapse of montmorillonite in the determination of external surface area by glycerol retention, the external surface area of several other associated minerals is seriously affected and a complete collapse of montmorillonite is not necessarily obtained. This pretreatment accordingly may introduce large errors in surface area measurements, and is therefore unsuitable.

On the other hand, the data show that pretreatment by saturation with the triethylammonium cation effectively prevents the adsorption of glycerol on the internal surfaces of expanding clay minerals, but does not alter external surfaces nor otherwise interfere with their normal adsorption of glycerol. The triethylammonium saturation thus permits accurate measurements of surface area, and is therefore suitable for general use with soils and clays.

Materials Studied

The clays and soils used in the study are described in table 1. They were selected to provide representatives of the principal silicate clay minerals, and also one of the common hydrous oxides.

A normal solution of triethylammonium chloride, used for saturating the soils and clays, was prepared by neutralizing triethylamine (Eastman) with hydrochloric acid to pH 7.0, and diluting with distilled water.

The retention of glycerol by the experimental materials was determined essentially as described previously (1). Briefly, the procedure as used here involved preparing dupli-

cate 1- to 3-gram specimens of material passing the No. 40 sieve; drying at 110° C. in aluminum foil dishes and weighing to 0.2 mg.; adding 10 ml. of a 2-percent aqueous solution of glycerol; and heating to constant weight at 110° C. in a mechanical convection oven containing a constant supply of glycerol vapor at a partial pressure just below saturation. The gain in weight represents the glycerol retained, and is expressed as a percentage of the 110° C. oven-dry weight.

Three determinations of retention were made for each material: (1) untreated, (2) previously heated to 600° C. for 4 hours, and (3) previously saturated with TEA. Saturation was accomplished by treating 10 grams of material with an amount of the triethylamine chloride solution sufficient to exceed the cation exchange capacity fivefold. The material and solution were thoroughly mixed, centrifuged, and the supernatant liquid decanted. This treatment produced strong flocculation with all of the materials studied, and gave very clear supernatant liquids with a minimum of centrifugation. After three such treatments, the material was thoroughly washed in the same manner with acetone, dried over a steam bath, and lightly pulverized.

Apparent values of external surface were calculated from the glycerol retained after either pretreatment by multiplying the percentage retained by the factor 17.65; the apparent internal surface in either case was computed as 35.3 times the difference between the percentage of glycerol retained by the untreated sample and the percentage retained by the treated sample (1).

Since all calculations were made on the basis of the 110° C. dry weight, specimen weights of the 600° C. heated material were corrected for the loss in weight between 110° C. and 600° C. A similar correction of the TEA saturated material to the same basis would involve adjustment for the increase in weight due to the excess weight of the triethylammonium cation over that of the cation it replaced. This correction was applied only to the montmorillonite samples, since it was considered negligible for the others due to

their considerably lower cation exchange capacities.

Test Results

Data for a comparison of the effects of the two pretreatments are given in table 1, including experimental values of glycerol retention and the corresponding calculated values of external surface area. For two of the clays, surface area values obtained by the Brunauer-Emmett-Teller (BET) method using nitrogen sorption are also included. In the following discussion, the experimental materials are grouped according to mineral composition.

Montmorillonite

The external surface area obtained for Southern bentonite after 600° C. heating agrees quite closely with that after TEA saturation. This implies that with this montmorillonite, neither pretreatment significantly alters the mineral surfaces or interferes with normal adsorption, and that both effectively prevent the entrance of glycerol molecules into interlayer spaces. These implications are substantiated by the good agreement between the results for the glycerol sorption and the BET methods. The area indicated for the latter method was determined by Lopez-Gonzalez and Deitz (3) on another specimen of the sample used in the present work.

With the Wyoming bentonite, on the other hand, it was necessary to heat to 700° C. to irreversibly collapse the lattice, and the external surface area found after heating to this temperature is about 5 m.²/g. lower than that after TEA saturation. If this is a significant difference, it is probably due to a slight sintering effect of the heat treatment at the higher temperature.

Kaolinite

Table 1 includes data for two kaolins. With the Edgar Plastic Kaolin (E.P.K.), TEA saturation resulted in a glycerol retention slightly lower than that of the untreated material. The reduction is thought to be due to a small quantity of montmorillonite,

Table 1.—Effect of pretreatments on external surface area

Sample No.	Material	Clay mineral type	Source	Glycerol retention ¹			External surface area		
				110° C. dried	600° C. heated	TEA saturated	Calculated from glycerol retention after—		From nitrogen adsorption, BET method
							600° C. heating	TEA saturation	
R-145.....	Southern bentonite ²	Montmorillonite.....	Panther Creek, Miss.....	<i>Pct.</i> 20.7	<i>Pct.</i> 4.55	<i>Pct.</i> 4.61	<i>m.²/g.</i> 80.3	<i>m.²/g.</i> 81.4	<i>m.²/g.</i> ³ 83.8
R-51.....	"Volclay" Wyoming bentonite.....	do.....	American Colloid Co.....	19.0	³ 2.56	2.85	³ 45.2	50.3	-----
R-164.....	Edgar plastic kaolin (E.P.K.) ⁴	Kaolinite.....	Edgar Clay Co.....	2.49	1.70	2.20	30.0	38.8	⁴ 35.6
R-165.....	Lustra kaolin ⁴	do.....	Southern Clay Co.....	.81	.72	.85	12.7	15.0	-----
R-152.....	Pithian illite.....	Illite.....	Pithian, Ill.....	6.1	3.5	4.5	62	79	-----
R-170.....	"Mountain leather" ⁵	Palygorskite.....	Metalline Falls, Wash.....	9.5	19.0	9.3	335	165	-----
S-61.....	Ando soil ⁶	Allophane.....	Japan.....	16.2	8.3	16.2	146	293	-----
PC-1110.....	Grizzly soil.....	Allophane, montmorillonite.....	B horizon; Crook County, Oreg.....	13.0 [*]	4.4	7.0	78	124	-----
PC-246.....	Linker soil.....	Gibbsite, kaolinite, soil chlorite.....	B horizon; De Kalb County, Ala.....	1.75	2.2	1.4	37	25	-----

¹ Each glycerol retention percentage is an average of values from duplicate specimens.

² Sample and nitrogen surface area value supplied through the courtesy of V. R. Deitz, National Bureau of Standards.

³ Sample heated to 700° C., since this temperature was required to secure irreversible lattice collapse.

⁴ Sample and nitrogen surface area supplied through the courtesy of W. C. Ormsby, National Bureau of Standards.

⁵ Sample supplied through the courtesy of E. B. Henderson, U.S. National Museum.

⁶ Sample supplied through the courtesy of L. D. Whittig, formerly of the U. S. Department of Agriculture.

amounting to about 1 percent. This mineral could not be identified in an X-ray diffraction examination of the whole material, but was definitely detected in a very fine fraction separated by centrifugation. On the other hand, a further and substantial reduction in glycerol retention is indicated for the 600° C. heat treatment. This is thought to be the result of a lowering of the surface area by the heat treatment itself, in accordance with the previously cited experience of Gregg and Stephens (4). It is also noteworthy that surface area values obtained from the glycerol retention are in good agreement with those given by the BET method for a sample from the same stock.

With the Lustra kaolin, TEA saturation did not result in a reduction in glycerol retention, which indicates that this material is virtually free of montmorillonite. The 600° C. pretreatment, however, produced a modest decrease in retention and surface area, again in agreement with the findings of Gregg and Stephens.

Illite

Saturation of Fithian illite with TEA reduced glycerol retention from 6.1 to 4.5 percent, which is interpreted as due to a collapse of expandable layers equivalent to a content of about 7 percent of interlayered montmorillonite. The indicated value of external surface area, 79 m.²/g., is in good agreement with values quoted by Brooks (9).

On the other hand, when the 600° C. heat treatment was used, retention was 3.5 percent, which corresponds to an external surface of 62 m.²/g. This effect closely parallels the findings of Orchiston (10), who noted a reduction from 78 to 57 m.²/g. on heating Fithian illite to this same temperature.

Palygorskite

The sample of "mountain leather" consists chiefly of palygorskite, along with a small content of quartz. No montmorillonite could be identified by X-ray diffraction, in contrast to the more familiar "Georgia attapulgite" which the authors have found to contain a considerable amount. The absence of montmorillonite from the present sample is further attested to by the fact that there is essentially no difference between the glycerol retention values of the untreated and the TEA saturated material.

In contrast to the negligible effect of the TEA saturation, the glycerol retention after 600° C. heating was more than double that of the untreated material. This indicates that the heat treatment produces a large amount of new surface area. This apparently contradicts the results obtained by McCarter et al. (11) with nitrogen adsorption measurements, which indicated no change in the surface area of the attapulgite form of this mineral on heating to this temperature. However, the results for the two adsorbates can be reconciled if the difference in molecular size is taken into account. These authors state that the dimensions of the structural channels which run parallel to the long axis of the fibrous particles are such that the

inner surfaces are already accessible to the nitrogen molecules in the unheated material. Therefore, the heating could not expose any additional surface of this type to nitrogen adsorption.

On the other hand, it is thought that glycerol molecules are too large to enter the channels of the palygorskite before it is heated, but the large increase in retention after heating suggests that they can then do so.

It is of interest to note that the external surface area, calculated from the glycerol retention of the TEA saturated palygorskite, corresponds approximately to what would be expected from the dimensions of the individual fibers of attapulgite reported in the literature. The value of 165 m.²/g. given in table 1 corresponds to an average fiber diameter of about 200 A.; examination of attapulgite in the electron microscope (12) has indicated that the fibers are about 50 to 100 A. thick and about 100 to 300 A. wide.

Allophane

An Ando soil from Japan was selected for studying the surface area of allophane. This soil was shown by X-ray diffraction to consist of noncrystalline material, except for a small content of quartz and feldspar. Its differential thermal analysis curve was identical to that of "allophane A" as described by Fieldes (13). The glycerol retention of the untreated material is almost as high as that of montmorillonites. Nevertheless, the fact that the retention is unchanged by TEA saturation is considered as further evidence that montmorillonite is not present, and that all of the surface area is external.

In direct contrast, the 600° C. pretreatment resulted in a reduction of retention to approximately half that of untreated or TEA saturated specimens. This drastic effect is thought to be due to sintering, but regardless of its cause, it clearly indicates that for soils and clays containing allophane, pretreatment at 600° C. would lead to very large errors in surface area.

Allophane-montmorillonite

The Grizzly soil was selected for study because X-ray diffraction indicated that it contains a large amount of amorphous material, along with an appreciable admixture of montmorillonite. Since this soil is the weathering

product of materials of volcanic origin, the amorphous constituent is assumed to be allophane. Differential thermal analysis supported this interpretation and gave no indication of the presence of free hydrous oxides of iron or aluminum.

The glycerol retention data given in table 1 are what would be expected for a material containing montmorillonite and allophane. The retention value of the untreated material is reduced from 13.0 to 7.0 percent by the effect of the TEA saturation on montmorillonite; however, with heat treatment alone a retention of 4.4 percent was obtained, which indicates some destruction of the surface area of the allophane constituent as a result of the heating. Accordingly, in estimating the external surface area of this soil, an apparently very low value of 78 m.²/g. is obtained after the heat treatment, whereas a more realistic value of 124 m.²/g. is given by TEA saturation.

The difference in retention of the untreated and the TEA saturated soil indicates a montmorillonite content of about 25 percent. This is substantially in accordance with the X-ray diffraction evidence.

Hydrous oxides in soils

The clay fraction of Linker soil, which was included to represent materials containing hydrous oxides, was shown by X-ray diffraction and different thermal analysis to consist largely of gibbsite, along with kaolinite and a soil chlorite as accessory minerals. A small reduction in the glycerol retention is noted for TEA saturation. If this is interpreted as due to internal surface, it corresponds to a little over 1 percent of expandable layers, presumably contained as mixed-layers in the soil chlorite.

In contrast to most of the other materials studied, the Linker soil was found to increase in retention as a result of the 600° C. treatment. This can be attributed to an increase in surface area from the decomposition of the gibbsite, in accordance with the previously cited work of Russell and Cochran (6).

An analogous effect was noted for a relatively pure mineralogical sample of lepidocrocite, which showed an increase in retention from 0.3 percent for the untreated material to 3.0 percent for specimens previously heated to 450° C. A sufficient quantity of the lepidocrocite was not available for a thorough study,

Table 2.—Effect of pretreatments on internal and total surface areas calculated from glycerol retention

Sample No.	Material	Clay mineral type	Internal surface area, calculated from retention after—		Total surface area, calculated from retention after—	
			600° C. heating	TEA saturation	600° C. heating	TEA saturation
			m. ² /g.	m. ² /g.	m. ² /g.	m. ² /g.
R-145.....	Southern bentonite.....	Montmorillonite.....	570	568	650	649
R-51.....	Wyoming bentonite.....	do.....	580	570	625	620
R-164.....	Edgar plastic kaolin.....	Kaolinite.....	28	10.2	58	49
R-165.....	Lustra kaolin.....	do.....	3.2	0	15.9	15.0
R-152.....	Fithian illite.....	Illite.....	92	56	154	135
R-170.....	"Mountain leather".....	Palygorskite.....	0	0	335	165
S-61.....	Ando soil.....	Allophane.....	278	0	424	293
PC-1110.....	Grizzly soil.....	Allophane, montmorillonite.....	304	212	382	336
PC-246.....	Linker soil.....	Gibbsite, kaolinite, soil chlorite.....	0	12.3	37	37

but the data for both of these materials indicate that the 600° C. heat treatment is clearly unsuitable for soils and clays containing the crystalline hydrous oxides of iron or aluminum.

Effect of Pretreatment on Estimation of Internal and Total Surface Areas

In addition to the effects of the 600° C. heat treatment on external surface areas, there are corresponding errors in the calculations of internal and total surface areas. Values of internal and total surface areas for both the 600° C. heating and the TEA saturation treatments were calculated from the data in table 1, and are presented in table 2. Internal surface areas were calculated by the method given earlier, and the total area is, of course, the sum of the internal and external areas.

From the data given in table 2, it is seen that for those samples having internal surface, the internal surface areas for the 600° C. treatment vary quite widely from those for TEA saturation, except for the montmorillonite samples. In estimating the total surface area, on the other hand, the method of calculation itself provides a compensating mechanism, so that differences for the two pretreatments are not large, except for the materials containing palygorskite or allophane.

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Nonelastic Deformations in Continuous Concrete Structures

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The performance of continuous-reinforced concrete bridges has been rather unsatisfactory because of the objectionable cracks and sags that develop in the spans following construction. This difficulty can be alleviated, however, by modifying the designs to compensate for the effect of the drying shrinkage of concrete. Creep and plastic flow of the concrete are also recognized as minor contributing factors to the problem of cracking and sagging.

Designing for shrinkage moments usually adds less than 1 percent to the cost of a structure. Heavier and longer lengths of negative moment reinforcing steel are required because of the increase in negative moments and because the points of inflection move farther out in the spans as a result of the increase in negative moments produced by the effect of drying shrinkage of the concrete. For some designs the concrete on the compression side of the girders will be overstressed in the vicinity of the interior piers. A slight change in the geometry of the girders usually reduces these stresses sufficiently and adds only a very small volume of concrete to the design. In box girders this can be accomplished by thickening the compression flange of each girder for a short distance at each interior pier.

CONTINUOUS-REINFORCED concrete bridge spans when designed by the elastic theory with the reinforcing steel stressed to 20,000 p.s.i. and the concrete stressed to 1,200 p.s.i. by combined dead load, live load, and impact moments, as permitted by the American Association of State Highway Officials' *Standard Specifications for Highway Bridges*, usually continue to deflect under dead load for several months after completion. This nonelastic deflection is accompanied by severe cracking of the floor slabs in the negative moment lengths of the girders. Some spans may sag sufficiently to impair the riding quality of the roadway surface and mar the appearance of the structure. Careful observance of continuous-reinforced concrete deck girder, I-girder, and box girder spans has shown that these defects may appear as early as 3 months after completion of the work. Some spans develop objectionable sag before highway live loads are placed on the structure.

Studies made to determine the cause of this behavior revealed that reinforced concrete simple spans continue to deflect for about 4 years after construction. The final deflection may become as much as four times the elastic

deflection under dead load at the time of centering removal. In long-span continuous-reinforced concrete deck girders, I-girders, and box girders designed for maximum allowable unit stresses under dead load, live load, and impact moments, the nonelastic deflection is accompanied by numerous cracks across the floor slabs in the negative moment areas of the girders. In the three-span continuous layouts with end spans less than 75 feet, most of the sag occurs in the middle span. In the longer spans appreciable sag occurs in all members. Some examples of cracking and sagging found in existing three-span bridge systems follow.

Measurements of Sags and Cracks in Completed Structures

A continuous box girder with 100-120-100-foot spans sagged 2 inches in the center span and $2\frac{1}{16}$ inches in the end spans after the bridge was opened to traffic. A 75-100-75-foot span continuous I-girder had $\frac{5}{8}$ - and $\frac{7}{16}$ -inch sags in the end spans and $\frac{1}{2}$ inch in the center span below theoretical finished grade, $2\frac{1}{2}$ years after completion. These spans had approximately $\frac{1}{2}$ -inch camber when completed. A 50-70-50-foot span continuous-reinforced concrete deck girder was about 8 months old before any live loads were placed on it. These spans cracked, and the middle span sagged $\frac{1}{2}$ inch under dead load when only 5 to 6 months old. Two identical continuous box girders with 75-100-75-foot spans were built to maximum camber ordinates of 1 inch in the end spans and $1\frac{1}{8}$ inches in the middle span. Approximately $\frac{1}{2}$ -inch camber remained when the structures were completed. Levels taken when the spans were 3 years old indicated $\frac{3}{8}$ - and $\frac{1}{4}$ -inch sags in the end spans and a $\frac{3}{4}$ -inch sag in the middle span, measured below theoretical finished grade.

One of the latter two bridges had typical cracks and sags. The cracks over the center of the interior piers and for about 5 feet on either side were on approximately 12-inch centers, their spacing increasing to about 2-foot centers, 20 feet on either side of the piers. A tool was made for use with a micrometer to measure the width of the cracks. The width of those on 12-inch centers varied very little. The cracks directly over the center of piers measured 0.0312 inch. The maximum elastic elongation of reinforcing steel in a 12-inch length equals $(32,000 \times 12)/30,000,000 = 0.0128$ inch. Assuming shrinkage of 0.0006 inch per inch in the concrete—neglecting the probability that the

concrete carries some tension and disregarding the effect of creep which tends to nullify the shortening effect of shrinkage—the concrete will shorten $0.0006 \times 12 = 0.0072$ inch in 12 inches. Then the maximum width of the cracks for these extreme and very improbable conditions should be 0.0200 inch.

In view of tension in the concrete and the creep effect, the concrete fiber shortening factor is probably less than 0.0003 inch over the center of piers. Then the maximum width of the cracks in concrete with reinforcing steel stressed to the elastic limit should not exceed 0.0164 inch. Under the theoretical dead load stress in reinforcing steel of about 12,000 p.s.i., the width of the cracks should not exceed 0.0088 inch. Therefore, the cracks over the center of piers measuring 0.0312 inch were nearly four times larger than they should have been under the theoretical existing stress ($0.0312 - 0.0088 = 0.0224$ inch). The widths of the cracks indicated that the negative moment reinforcing steel had stressed beyond the elastic limit.

One continuous-reinforced concrete deck girder bridge with 60-70-60-foot spans was found which had no perceptible sag in any span and there were no cracks in the floor slabs. This bridge had been in place for about 4 years. The spans were carefully analyzed and it was found that the theoretical stresses in the reinforcing steel and the concrete were very low in the parts of the girders subject to negative moment.

The data developed from these investigations indicated clearly that the dead load negative moments were increasing with time and that the rate of increase paralleled known rates of increase in drying shrinkage of aging concrete. It was concluded that the increase in dead load negative moments was produced by a gradual change in the physical shape of the continuous system, primarily through the effect of drying shrinkage in the concrete. It was recognized that creep and plastic flow could be minor contributing factors. The study indicated that perceptible sag developed in spans only when the negative moment reinforcing steel was stressed beyond the elastic limit or when the concrete was stressed into the plastic flow range.

Modification of Bridge Design

A method of determining the probable mechanical effect of drying shrinkage of concrete in continuous concrete bridge spans was developed by the author in 1955. The new theory is explained in subsequent sections of

this article. The studies leading to the development of the theory were begun in 1952 after engineers of the Alabama State Highway Department and the Bureau of Public Roads became concerned with the safety of the 100-120-100-foot continuous box girder spans mentioned earlier and other continuous concrete spans. The State Bridge Engineer was considering abandoning this type of design on future State and Federal-aid bridge projects unless a solution to this problem could be found.

After observing sags and cracks in the spans of two structures—an 80-100-80-foot span continuous concrete I-girder and a 50-70-50-foot span reinforced concrete deck girder—the designs were modified by the State highway department to agree with the theory of increased dead load negative moments due to drying shrinkage in the concrete. Two I-girder type bridges of the modified design were completed on Federal-aid projects early in 1956. The girders, in the negative moment lengths, were designed for combined dead load, live load, impact, and shrinkage moments using a shrinkage factor of 0.0006. Levels taken periodically at one of the bridges disclose no measurable increase in dead load deflections since removal of the falsework, nor have cracks been observed in the negative moment lengths of either set of spans. Another bridge of the deck girder type, similar to the one that had cracked and sagged under dead load, has been built using the modified design. After being in use 3 years, no cracks or sags had developed.

Twelve existing designs have been modified and fifty new three- and four-span reinforced concrete deck girder and I-girder standard designs have been prepared by the State highway department for use on Interstate System grade separation structures and other State and Federal-aid bridge projects. Several continuous box girder designs for use on Interstate System grade separation structures have been prepared by a consultant. All girders, in the negative moment lengths, were designed for combined dead load, live load, impact, and shrinkage moments using a factor of 0.0006. Unit stresses for the combined moments do not exceed the allowable unit stresses permitted by the AASHTO specifications. Twelve Interstate System grade separation structures and several bridges over streams, constructed according to the modified designs, have been completed for several months; none of the spans have shown a tendency to crack or sag.

Designing for shrinkage moments adds very little to the cost of a structure, usually less than 1 percent. The quantity of negative moment reinforcing steel is increased some due to the additional requirements caused by the increase in negative moments and to the additional lengths needed because the points of inflection move further out in the spans. In some designs the concrete in the compression flange will be overstressed over a short length of the girders in the immediate vicinity of the interior pier supports. Modifying the geometry of the girders throughout these lengths to reduce the concrete stresses usually requires a very small amount of additional

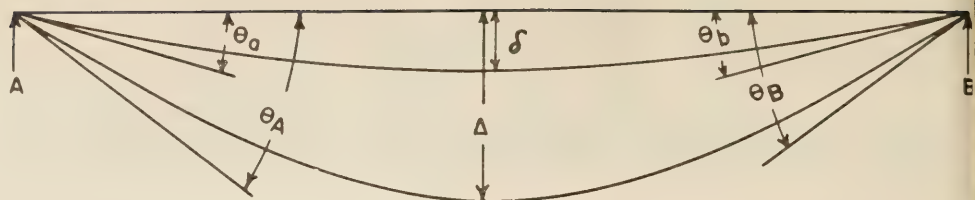


Figure 1.—Relation of rotations of a concrete beam at the end supports to deflections at the center of the span.

concrete. An extra short compression reinforcing bar or two will reduce the concrete stresses sufficiently in some designs.

Development of Theory

Nonelastic deflection of a concrete beam is produced by nonelastic fiber deformations within the beam just as elastic deflection is produced by elastic fiber deformations. Nonelastic fiber deformations in concrete may be produced by three separate actions: shrinkage due to the concrete setting and drying, creep due to dead load stresses, and plastic flow in areas of very high stresses (1).¹ The shrinkage effect is much greater initially than the effects of creep and plastic flow. The drying of concrete is more complete in thin sections; therefore, more shrinkage occurs in a thin section such as a floor slab than in the thicker sections of a beam. The shrinkage effect on the deflection of a span probably starts when the concrete is only a few hours old or near the time it enters the final set period.

Shrinkage, creep, and plastic flow

Shrinkage in thin sections of concrete develops at a fairly rapid rate for the first few months and continues at a decreasing rate for an indefinite period. Most of the shrinkage develops in the first 8 or 9 months.

Creep, when separated from shrinkage, is a delayed deformation similar to elastic deformation and is produced by the same stresses causing elastic deformation. It is a continuing action over an indefinite period. At working stresses, creep is proportional to stress. The effect of creep may be taken into account by reducing the value of the modulus of elasticity of the concrete. The stress in the reinforcing steel is increased through an increase in the value of n , the ratio of the modulus of elasticity of the steel to the modulus of elasticity of the concrete.

Plastic flow in concrete is that deformation which occurs at very high stress (above 70 percent of ultimate), the concrete acting as a viscous solid. Plastic flow deformations are not proportional to stress. If plastic flow occurs in continuous concrete spans, it will probably be at points of high concrete stress produced by the combined dead load, live load, impact, and shrinkage negative moments. Plastic flow at these points will produce a loss in continuity in the spans, which in turn causes the dead load deflections to increase.

In thin concrete members, such as floor slabs, the shrinkage factor due to the concrete setting and drying may be as high as

0.0006 (1, 2, 3). In the lightly reinforced top flange portions of a beam there is little or no opposition to concrete shrinkage. Shrinkage in this flange will temporarily relieve the concrete of carrying a portion of its compression load, causing the reinforcing steel to take more compression until it is stressed beyond its elastic limit; the steel is then deformed and the concrete again acts to carry its proportion of the compression load. Thus, the shortening of the top flange lengths is equal to the shrinkage in the concrete. The bottom flange of a beam is heavily reinforced in the positive moment areas. Shrinkage will increase the tension stresses in the concrete and tend to cause cracking. Creep will tend to nullify shrinkage in the bottom flange. Shrinkage will have little or no effect on the deformations within the reinforcing steel or the length of a given portion of the bottom flange of the beam.

Rotation of beams at end supports

A simple span beam cannot deflect from any cause without producing rotation at the supports. The amount of rotation at the supports, as well as the deflection at any point along the span, is the summation of the effect on rotation or deflection of each of the individual changes in fiber lengths, regardless of what produces these changes. Since shrinkage may be assumed to occur uniformly along the positive moment lengths of a beam, the shape of the final deflection curve will be more nearly circular than parabolic as in elastic deflection (4). Therefore, the ratios of the final rotations θ_A and θ_B (fig. 1) at the beam ends to the elastic ratios θ_a and θ_b , respectively, will be greater than the ratio of the final deflection, Δ , to the elastic deflection, δ , at the center of the span. If Δ equals 4δ , $\theta_A > 4\theta_a$ and $\theta_B > 4\theta_b$.

In the simple span the rotations, θ_A and θ_B , have no effect on the stresses within the beam. However, in the indeterminate structure (fixed-end spans and continuous spans), the effect of this additional rotation on the stresses of the beam may be tremendous. The bending moment pattern is governed by the end rotations of the spans, considered as simple spans. In the conjugate beam method of stress analysis, the fixed-end moments of a beam can be determined directly from the end rotations. In the fixed-end moment formula $M_{AB}^F = \theta_A K_A - C_{BA} \theta_B K_B$, it can be seen that increasing the end rotations will increase the fixed-end moments proportionally. K_A and K_B , the Cross stiffness factors, and C_{BA} , the Cross carryover factor, are fixed by the moment-of-inertia pattern of the beam which does not change.

¹ Italic numbers in parentheses refer to the list of references on p. 196.

In the continuous span, the fixed-end moment distribution factors are determined from the Cross carryover and stiffness factors of the beam. These factors are not affected by fiber deformation. Therefore, the negative moments of a continuous span are in proportion to the fixed-end moments, and nonelastic deformations from shrinkage in the positive moment lengths of a beam will increase the negative moments. When combined dead load, live load, impact, and shrinkage moments are sufficient to stress the negative moment reinforcing steel beyond the elastic limit, the reinforcing steel will undergo nonelastic elongation and the concrete will crack.

Nonelastic elongation of steel

A very slight nonelastic elongation of the reinforcing steel has two immediate effects. It causes a portion of the continuity of the spans to be lost which increases the end reactions and, at the same time, causes a shifting of the neutral axis of the beam away from the reinforcing steel, thus increasing the effective depth, jd , and reducing the stress in the reinforcing steel. Restoration of a portion of the reactions lost through the initial effect of shrinkage reduces the moments produced by shrinkage. The stress in the concrete over the interior piers is increased by the increase in negative moments through the effect of shrinkage. The shifting of the neutral axis of the beam produced by yielding of the reinforcing steel further increases the concrete stress. Plastic flow which may occur from the high stresses now existing in the concrete over the interior piers will cause a further loss of continuity and an increase in the end reactions.

In some spans the loss of end reaction through the initial effect of shrinkage may be more than completely restored. The spans will then act partially as simple spans. These actions may temporarily stabilize under dead load with the reinforcing steel stresses at the elastic limit of the steel. Then as live loads come onto the spans, the negative moments produced by the live loads will cause yielding in the steel. Since yielding in the reinforcing steel has the immediate effect of reducing the stress producing the yield, it can be seen that the nonelastic elongation of the steel will be small. The modulus of elasticity of the steel probably increases as the steel yields, thus further limiting the size of the cracks in the concrete.

This chain action started by shrinkage in the concrete will stabilize at some point depending on the design. In designs with sufficient negative moment reinforcing steel to hold the steel stress below the elastic limit and the concrete stress below the plastic limit, the action ends with the shrinkage in the concrete and with only a small change in deflections through the actions of creep and shrinkage. In most bridge spans designed to maximum allowable stresses by the elastic theory of stress analysis, cracking of the slabs in the negative moment areas will first develop, thus permitting a small increase in deflections. These cracks may appear as early

as 3 months after pouring. Then plastic flow may occur in the concrete at points of high negative moments produced by shrinkage and through an increase in concrete stress caused by the shifting of the neutral axis. The loss of continuity through these actions increases the positive moments and thus increases the dead load deflections.

In spans designed for dead load stresses only, shrinkage may increase the negative moments sufficiently to move the points of inflection beyond the cutoff points of the negative moment reinforcing steel and cause collapse of the spans. The nonreinforced concrete may crack due to its inability to carry the tension stress imposed. The cracking of the beams reduces the areas available to resist shear. The second action leading to collapse of the spans is shear failure through the reduced areas (5).

Application of Theory

The increase in negative moments from nonelastic deformations results from a reduction in the dead load end reactions (6). This reduction in the end reactions can be deter-

mined by considering the beam as weightless and applying a shrinkage factor of e to the lightly reinforced top flange portions of the beam. Let c equal the depth of the beam. Thus, $e/c=M$. Then apply M as a uniform load over the lightly reinforced compression flange lengths of a conjugate beam (7), as shown in figure 2(a).

Three-span bridge system

Deflection Y_2 at the ends of the spans equals the moment under loads M of the conjugate beam at R_2 (fig. 2(b)) in a symmetrical system of spans. The curve represents the new physical shape of the unloaded system of spans after shrinkage has occurred, and Y_2 is the distance the ends of the girders would rise off the bearings at R_1 and R_4 if the girders were weightless. The effect on dead load moments within the system caused by this change in physical shape is the same as would occur if piers R_1 and R_4 each settled an amount equal to Y_2 .

Forces P are applied at the beam ends to reduce deflection Y_2 to zero, as shown in figure 2(c). P is the reduction in dead load end reaction.

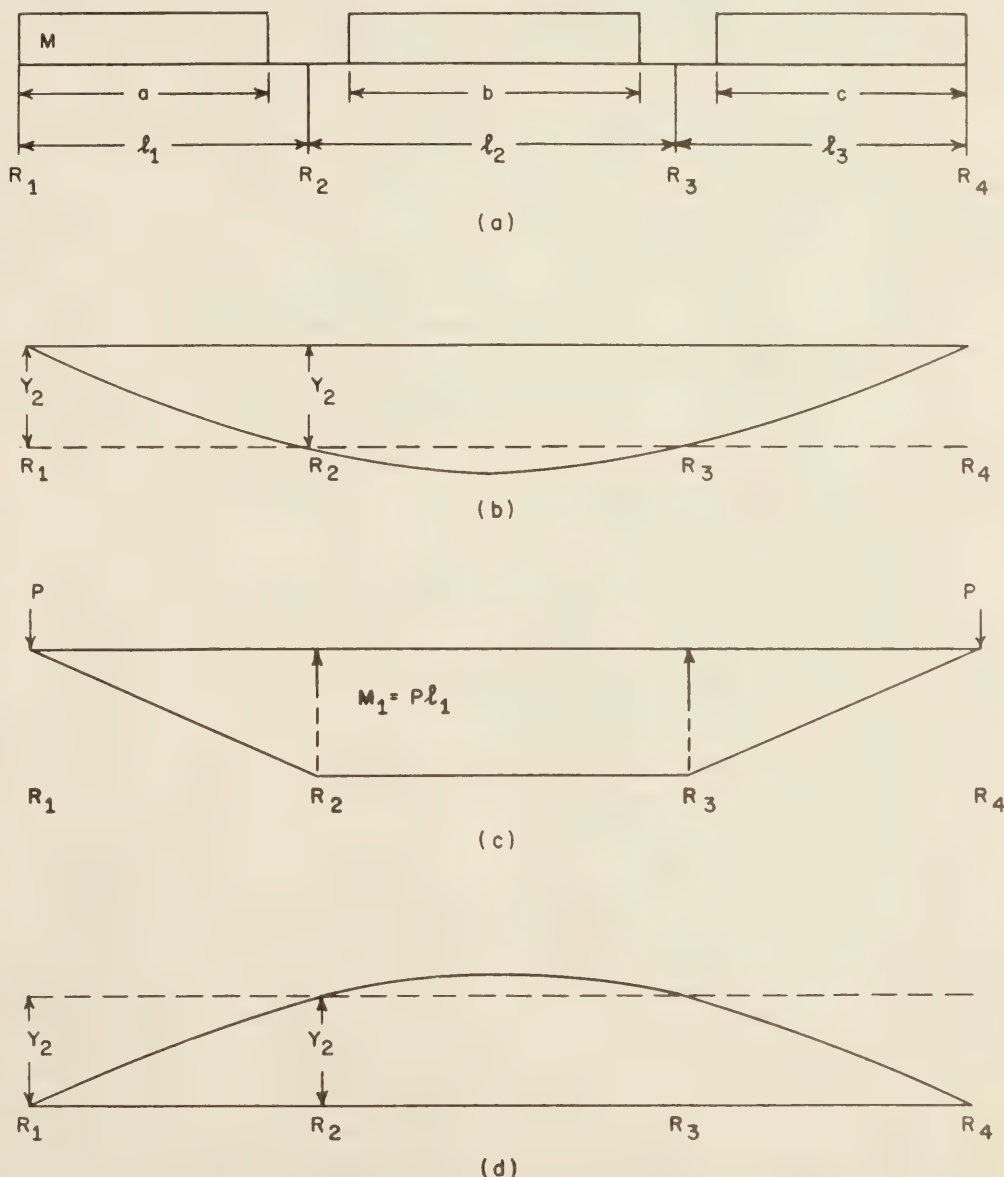


Figure 2.—Theory illustrated for a three-span bridge system.

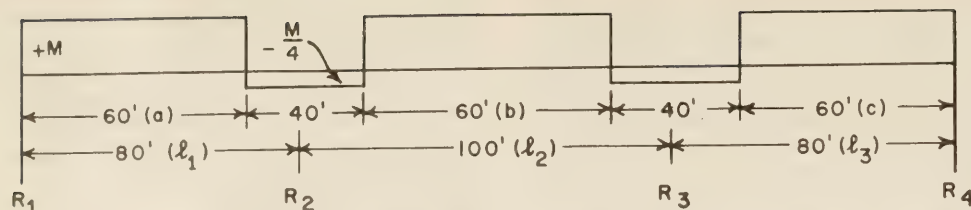


Figure 3.—Illustration of a conjugate beam loaded with positive and negative nonelastic weights.

The deflection Y_2 , indicated in figure 2(d), at the ends of the spans produced by loads P equals the moment of the conjugate beam at R_2 under loads M_1 divided by EI . (E =modulus of elasticity of the concrete.) For constant moment of inertia I ,

$$Y_2 = \frac{2Pl_1^3 + 3Pl_1^2l_2}{6EI} \quad \text{Formula 1}$$

When the system of spans is not symmetrical, l_1 does not equal l_3 or the variation in moments of inertia is not symmetrical about the center of the system, shrinkage deflections Y will have to be calculated at both R_1 and R_4 . These deflections are then treated the same as similar deflections produced by pier settlements.

As an example, in an 80-100-80-foot continuous I-girder designed for shrinkage forces, the depth of the beam c through the lightly reinforced compression flange lengths was 61.25 inches. Lengths a , b , and c (figs. 2(a) and 3) were 60 feet, $e=0.0006$, $M=e/c$, $I=685,299 \text{ in.}^4$, and $E=5,000,000 \text{ lb./sq. in.}$

$$Y_2 = 90M \times 80 - 60M \times 50 = 4,200M$$

$$Y_2 = \frac{4,200 \times 0.0006 \times 144}{61.25} = 5.92 \text{ inches}$$

From formula 1:

$$5.92 = \frac{2P(80)^3 + 1,728 + 3P(80)^2 \cdot 100 \times 1,728}{6 \times 685,299 \times 5,000,000}$$

$$P = 23,924 \text{ pounds.}$$

The negative moment due to P equaled 1,914 ft. kips. The dead load negative moment equaled 1,582 ft. kips; hence, the negative moment due to shrinkage was 121 percent of the dead load moment. The allowable stress in the negative moment reinforcing steel for combined dead load, live load, impact, and shrinkage moments was 20,000 p.s.i.

If more accuracy is desired in making these calculations, the shrinkage in the compression flange of the negative moment lengths and the variation in the moment of inertia of the beam may be considered. The following deflection formula which takes into account the variation in moment of inertia in a three-span symmetrical system may be used in calculating the value of P .

$$Y_2 = \frac{(\theta_{bb} + \theta_{bb2} + \theta_{cb})Pl_1^2}{EI_0} \quad \text{Formula 2}$$

In this formula θ_{bb} is the rotation at R_2 of span 1 with a one foot-pound moment at R_2 ; θ_{bb2} is the rotation at R_2 of span 2 with a one foot-pound moment at R_2 ; and θ_{cb} is the

rotation at R_2 of span 2 with a one foot-pound moment at R_3 , with each span considered as being a simple span and with the EI values reduced in the ratio of $EI_0=1$. I_0 is the smallest moment of inertia in the system. The values within the parenthesis are obtained from their respective moment diagrams after the moment ordinates have been reduced by dividing them by the appropriate value of I/I_0 . The depth of the beam over interior piers was 81.25 inches. The haunches were on parabolic curves and were 20 feet long. For simplicity in calculating the warpage of the beam due to shrinkage, the depth of the beam over these lengths will be considered as being constant. The compression flange was 24 inches wide and contained 10.92 square inches of reinforcing steel. In view of the thickness of the flange, its less exposed position, and the amount of reinforcing steel, the shrinkage factor e is probably about 0.0002 (2). The negative M ($M=e/c$) is about one-fourth of the M in the positive moment lengths.

The lengths a and c , figures 2(a) and 3, are approximately the lengths between the ends of the spans and the points of dead load contraflexure in spans 1 and 3. The length b is approximately the length between these points in span 2. The locations of the dead load points of contraflexure obtained by the elastic theory will move when the shrinkage effect is added to the elastic behavior of a beam.

$$Y_2 = 80M \times 80 - 60M \times 50 + 20 \times \frac{M}{4} \times 10 = 3,450M$$

$$Y_2 = \frac{3,450 \times 0.0006 \times 144}{61.25} = 4.87 \text{ inches}$$

From formula 2:

$$4.87$$

$$= \frac{(21.209 + 27.75 + 15.95)P \times (80)^2 \times 1,728}{5,000,000 \times 685,299}$$

$P=23,246$ pounds; the negative moment due to P is $23,246 \times 80=1,859,680$ foot pounds.

The shrinkage factor e on the compression side of the negative moment lengths will be smaller than at the positive moment lengths because of the thicker concrete sections and less exposure to drying. These sections usually have compression reinforcing steel which by the elastic theory carries low stress. Shrinkage in these sections will have the initial effect of forcing the steel to carry more of the compression load. Apparently the steel is not stressed beyond the elastic limit in the

early stages of the action. It may be stressed to this point if plastic flow occurs. When the compression reinforcing steel with the added stresses caused by shrinkage in the concrete is stressed below the elastic limit, shortening of the flange will result from the elastic shortening in the reinforcing steel through the action of these added stresses. The factor e will change as the bending moments produced by dead loads and shrinkage effect changes in the lengths; it will have a very small value at points with zero bending moment and will approach the value of the shrinkage factor of the concrete at points with high bending moments.

The beams are usually about twice as deep over the interior supports as they are in the positive moment lengths. A unit shortening of concrete fibers at these points would have only about half the effect of deforming the physical shape of the beam as would the same shortening at the thinner sections. Shrinkage in these sections will reduce the value of Y_2 (fig. 2(b)). However, with the value of negative M ($M=e/c$) being only about one-fourth of the value of M at lengths a , b , and c in figure 2(a), its effect on the value of Y_2 is small. The assumed points of inflection, which are the dividing points between positive and negative nonelastic loads, used in making these calculations must agree with the actual points of inflection of the beam under the combined effect of dead loads and shrinkage. It is usually necessary to make three or four trial calculations, adjusting these assumed points of inflection, until this agreement is reached.

In view of the difficulty at this time of estimating the ultimate value of the shrinkage factor e , which is affected by many factors including the cement and aggregate used, the proportions of the mix (especially the amount of water used in the mix), the curing of the concrete, and the climate, negative M is usually not included in the calculations. The additional cost of including the shrinkage effect over lengths a , b , and c (fig. 2(a)) in the design is usually small, and this procedure is on the side of safety.

Four-span bridge system

The procedure used in calculating the reduction in dead load end reactions produced by shrinkage in a four-span symmetrical system is similar to that used in the three-span system.

In figure 4(a), nonelastic loads M are applied to the span system considered to be a conjugate beam supported at R_1 and R_5 . Deflections Y_1 and Y_2 (fig. 4(b)) equal the moments in the conjugate beam at R_3 and R_2 , respectively, produced by the nonelastic loads M shown in figure 4(a). Deflection Y_3 equals Y_1 minus Y_2 . Deflection Y_1 represents the amount the beam, if weightless, would rise off the bearings at R_1 and R_5 . Y_3 is the amount the beam would rise off the bearings at R_2 and R_4 . These deflections may be treated as settlements in piers R_1 , R_2 , R_4 , and R_5 .

Forces P_1 (fig. 4(c)) are applied at R_1 and R_5 to reduce deflection Y_3 to zero. Y_3 equals

Y_5 minus Y_4 (fig. 4(d)). For constant moment of inertia,

$$Y_3 = \frac{P}{6EI} (3l_1l_2^2 + 2l_2^3) \quad \text{Formula 3}$$

Deflection Y_6 (fig. 4(e)) is the deflection remaining at R_1 and R_5 after loads P_1 have been applied. Y_6 equals Y_1 minus Y_5 . It is necessary to apply a second load P_2 at each end to reduce Y_6 to zero. The value of P_2 is found by loading the conjugate beam as illustrated in figure 4(f) and equating the moment at R_3 divided by EI to Y_6 . The reduction in dead load reactions at R_1 and R_5 equals $P_1 + P_2$. The increase in negative moment at R_2 equals $-(P_1 + P_2)l_1$. The increase in negative moment at R_3 equals $-P_1(l_1 + l_2) + C_{BC}P_2l_1$. C_{BC} is the Cross carryover factor from end B to end C of span 2.

General Considerations

The shrinkage effect does not become appreciable until after the spans are several weeks old. Therefore, to prevent overstress in the positive moment areas of the span during the early life, these areas should be designed for dead load, live load, and impact moments only. The concrete stress for combined moments including the shrinkage effect should not exceed allowable limits. Since shrinkage on drying is governed mainly by the unit water content of the mix, the driest mix feasible should be used in pouring continuous concrete spans.

The tensile stress at the roadway surface will exceed the ultimate tensile strength of the concrete throughout most of the negative moment lengths. The elastic elongation of the negative moment reinforcing steel over these lengths will be distributed in a few large cracks in areas where the concrete is not reinforced. To prevent such large cracks from forming, a portion of the negative moment reinforcing steel should be distributed across the floor slabs between girders. The reinforcing steel should be placed in the upper portion of the floor slabs and as near to the surface of the floor as the main slab reinforcing and the specifications will permit; otherwise objectionable cracks may develop from the normal elastic elongation of the reinforcing steel.

Deflection curves and camber diagrams calculated by the theory of combined elastic and nonelastic deformations are affected very little by the additional effect of shrinkage. The reduction in dead load end reactions as a result of shrinkage produces deflections opposite in direction and approximately equal to those produced directly by shrinkage in the spans.

Pouring Sequence of Concrete

The order of concrete pouring for a three-span system was designed to reduce the hazard of loss through a washout of falsework. The construction joints are placed approximately at dead load points of inflection. The pours are made in the alphabetical sequence indicated in figure 5. Sections A

plus B and C plus D are designed to be self-supporting after sections B and D are a few days old. This procedure reduces the hazard

of loss to a maximum of two sections and reduces the vulnerable periods to a total of 18 to 20 days. Pours A plus B made in warm

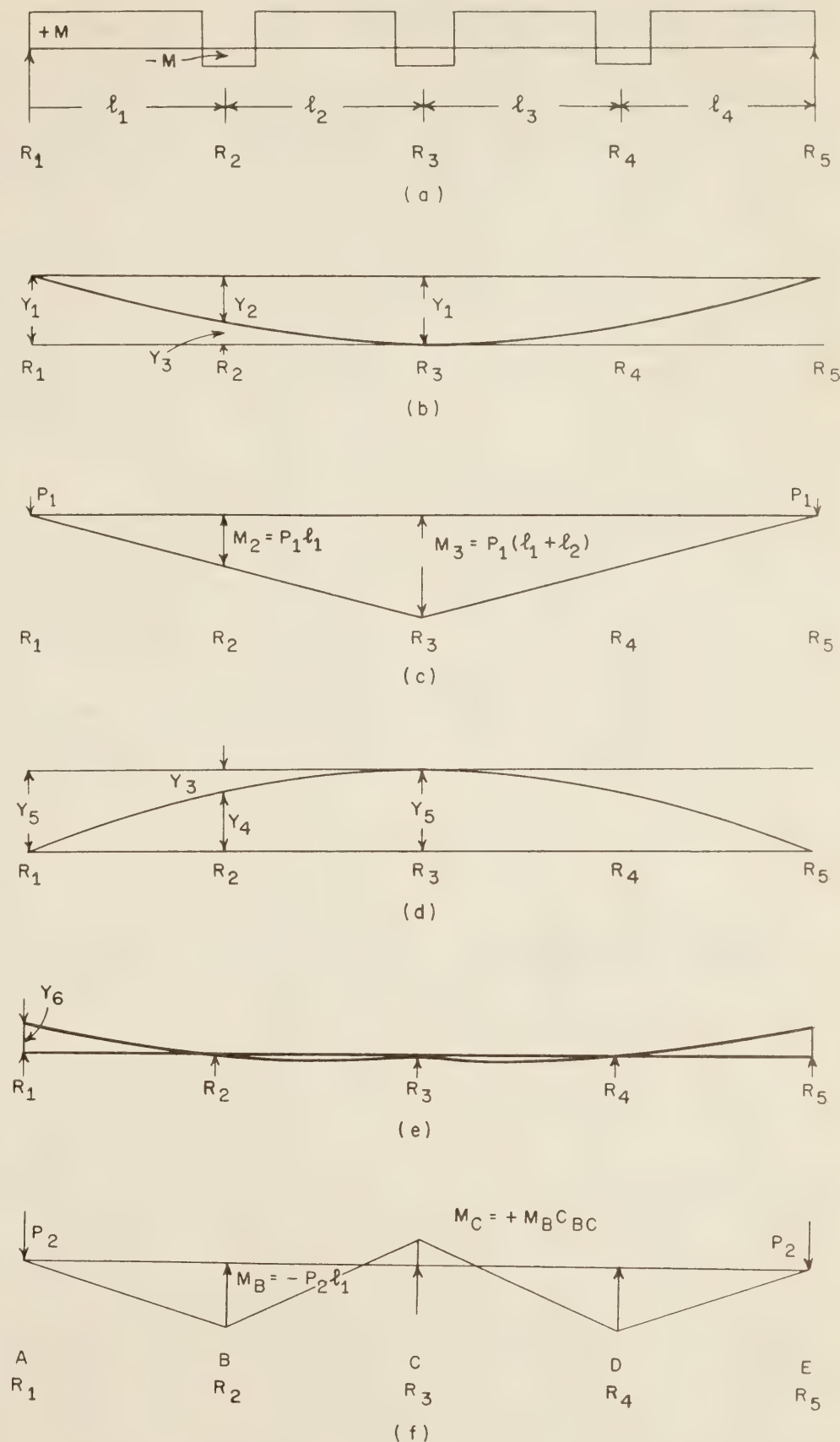


Figure 4.—Theory illustrated for a four-span bridge system.

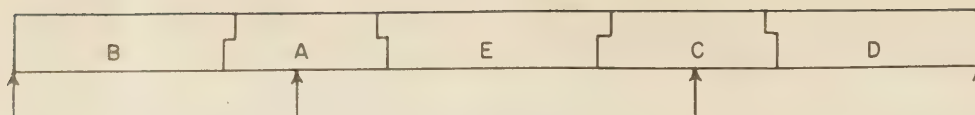


Figure 5.—Sequence of pouring concrete for a three-span bridge system.

weather would probably stand without damage after pour B was about 4 days old.

This pouring sequence allows initial shrinkage in the concrete in sections A, B, C, and D to develop before the spans are connected. However, it is believed that shrinkage in sections B, D, and E, which develops while the concrete is supported by the forms, either produces microscopic cracks in the concrete or places the concrete in tension. These microscopic cracks or tension act to change the physical shape of the beam when the supports are removed and have the same effect on stresses as shrinkage which develops after the spans are completed.

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(1) *Concrete Manual*. U.S. Department of the Interior, Bureau of Reclamation, sixth ed., 1955, ch. 1, par. 8, 11, and 14.

Figure 7, page 17, and figure 17, page 35, of the manual show a drying shrinkage factor of about 0.0006 for a mix containing 325 pounds of water per cubic yard of concrete. This is approximately the mix used in the structures discussed in the present study.

(2) *Concrete for Prestressing*, by Robert F. Blanks, Chief, Research and Geology Division, Bureau of Reclamation. Proceedings of the First United States Conference on Prestressed Concrete, Massachusetts Institute of Technology, Cambridge, Mass., Aug. 1951.

Shrinkage in 6-inch members (fig. 3, p. 145) may be expected to reach 0.0006 between 15 days and 9 months of concrete age. Figure 2 (p. 145) indicates 0.00065 shrinkage in 4-by-4-by-30-inch prisms between 3 and 9 months.

(3) *Study of Shrinkage in Concrete Frames*, by Morgan B. Klock and Robert R. Sheridan. Journal of the American Concrete Institute, vol. 28, No. 8, Feb. 1957, pp. 791-796.

The measured vertical shrinkage factor of a concrete building frame over a 16-year period was 0.000985. Horizontal shrinkage in reinforced concrete lengths was 0.00046. Figure 1 (p. 793) shows that the drying shrinkage factor in the building slabs was 0.00075 at a concrete age of 4 years or about 0.0005 at the end of 1 year.

(4) *Warping of Reinforced Concrete Due to Shrinkage*, by Alfred L. Miller. Journal of the American Concrete Institute, vol. 29, No. 11, May 1958, pp. 939-950.

(5) *Rigid-Frame Failure Raises Questions as to Standard Design Assumptions*, by Edward Cohen. Civil Engineering, vol. 26, No. 2, Feb. 1956, pp. 45-50.

Discussion of the collapse of about 4,000 square feet of roof at the Special Air Material Command Warehouse at Wilkins Air Force Depot, Shelby, Ohio.

(6) *Plastic Flow (Creep) of Reinforced Concrete Continuous Beams*, by G. W. Washa and P. G. Fluck. Journal of the American Concrete Institute, vol. 27, No. 5, Jan. 1956, pp. 549-561.

The authors indicate in conclusion No. 3 (p. 561) that the mid-span reaction of the two-span continuous beams increases about 5 percent. These beams carried a superimposed uniform load in addition to the dead load of the beams. This load should be considered as a live load and its effect on the reactions should be removed when changes in dead load reactions are being considered. When the effect of superimposed load on the type Y beams is removed, the increase in mid-span dead load reaction is about 12.4 percent. The decrease in dead load end reactions is 20.5 percent and the increase in the dead load negative moment over the mid-span support is 61.5 percent.

In one of the type X beams the increase in dead load negative moment is 52 percent. A shrinkage factor of 0.0006 applied over the positive moment lengths and 0.0003 over the negative moment lengths of these beams gives approximately these same increases in dead load negative moments. Figure 5 (p. 556) shows that most of the increase in mid-span reaction developed during the first 200 days. The curve showing this increase in reaction with time is similar to curves showing increase in shrinkage with time.

(7) *Strength of Materials, Part I, Elementary Theory and Problems*, by S. Timoshenko. D. Van Nostrand Company, Inc., New York, third ed., ch. 5, pp. 137-175.

Reports to Congress

Three reports have been submitted to the Congress by Secretary of Commerce Lewis L. Strauss during 1959. The titles of the reports are the *Progress Report on the Federal-Aid Highway Program*, the *Third Progress Report on the Highway Cost Allocation Study*, and *The Federal Role in Highway Safety*.

The *Progress Report on the Federal-Aid Highway Program*, prepared by the Bureau of Public Roads pursuant to the requirement of section 116 of the Federal-Aid Highway Act of 1956, describes the progress of the Federal-aid highway program in the 2½ years since passage of that act. Of particular interest is information on the status of the National System of Interstate and Defense Highways, presented in tabular form and on two large maps printed in color.

This report, printed as House Document No. 74, is for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 70 cents a copy.

The *Third Progress Report of the Highway Cost Allocation Study* was undertaken by the

Bureau of Public Roads pursuant to the requirement of section 210 of the Highway Revenue Act of 1956.

The purpose of the highway cost allocation study is to make available to the Congress information on the basis of which it may determine what taxes should be imposed by the United States, and in what amounts, in order to insure, insofar as practicable, an equitable distribution of the tax burden among various classes of persons using the Federal-aid highways or otherwise deriving benefits from them. The final report of the study is to be completed in January 1961.

The 66-page third progress report presents in summary form the information obtained from extensive studies of the numbers, types, and fuel consumption of motor vehicles registered, and the types, volumes, and weights of motor-vehicle traffic on the several highway systems. Forecasts of vehicles and travel are included. The report also presents a digest of a series of studies of the economic impact of highway improvement.

This report, printed as House Document No. 91, is available from the Superintendent of Documents at 35 cents a copy. Also available

from the same source is the 131-page First Progress Report, dated February 28, 1957, which presents a wealth of background material and a statement of the problem and plans for the study (35 cents per copy). The 6-page Second Progress Report, dated February 28, 1958, reported very briefly the nature of work underway.

The Federal Role in Highway Safety has been printed as House Document No. 93 and is for sale by the Superintendent of Documents at 60 cents a copy. This study, undertaken by the Bureau of Public Roads pursuant to section 117 of the Federal-Aid Highway Act of 1956, was made to determine what actions could be taken by the Federal Government to promote the public welfare by increasing highway safety.

The study evaluates the dimensions of the traffic accident problem and the relation of the driver, vehicle, and highway in connection thereto. It also evaluates the current safety activities and presents an adequate safety program. The findings give support to judgments formed over the past years on many highway safety matters, and also exposes widely held misconceptions of traffic accidents.

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